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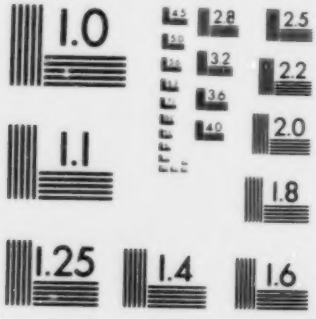

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STEPHENVILLE NFLD.

BREAKWATER MODEL STUDIES

December 1985

Public Works Canada

STEPHENVILLE, NEWFOUNDLAND

BREAKWATER MODEL STUDIES

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BREAKWATER MODEL STUDIES

Prepared for:

Transport Canada
Newfoundland Region
St. John's, Newfoundland

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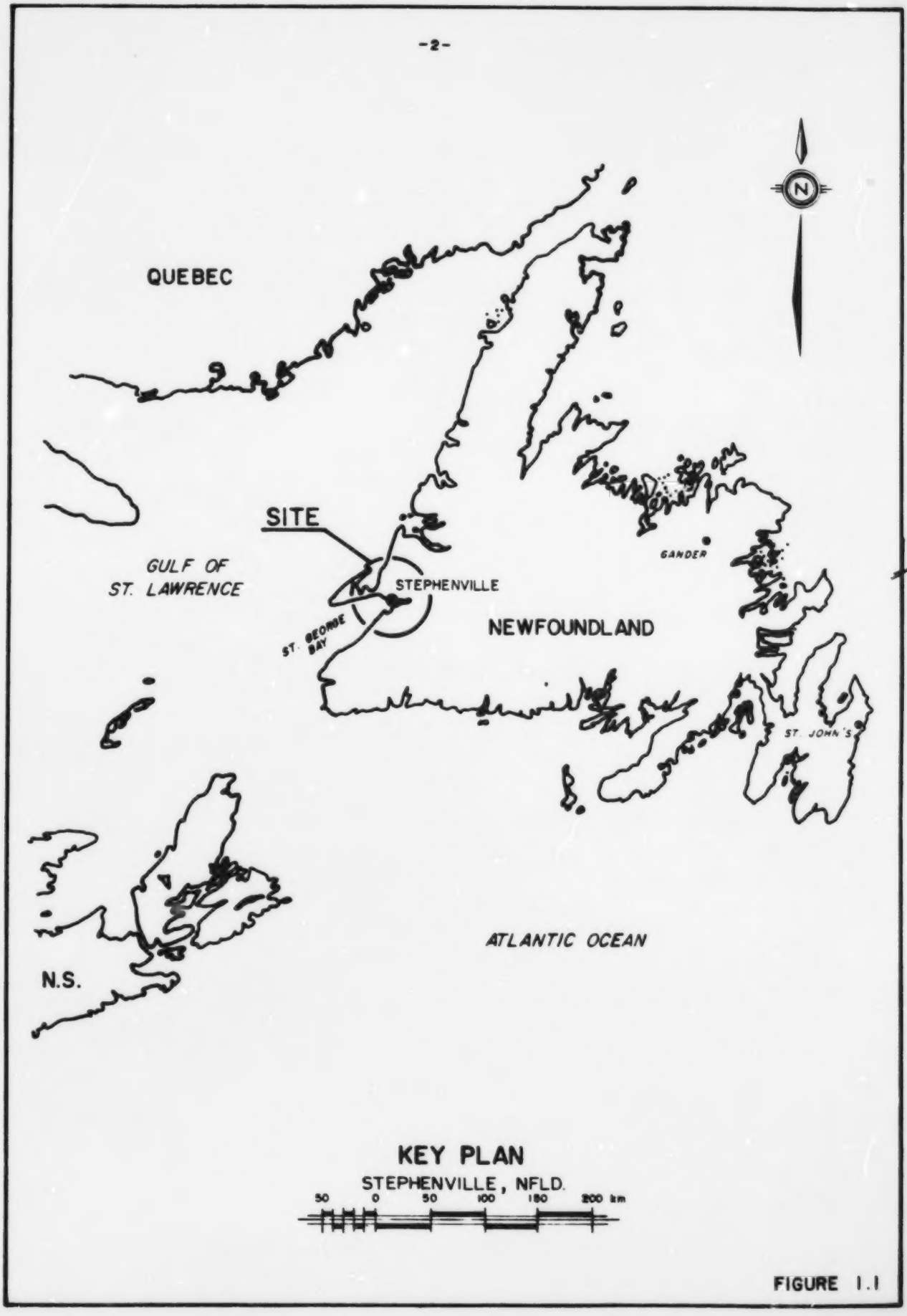
I INTRODUCTION

In March 1985, Transport Canada entered into a specific service agreement with Public Works Canada (Atlantic Region) to conduct breakwater model studies for Stephenville, Newfoundland (Figures 1.1 and 1.2). The studies consisted of a programme of two and three dimensional model breakwater tests to investigate the repairs required to the damaged head on the existing breakwater, and a programme of three dimensional tests to investigate proposed designs for the breakwater on the south beach. These tests were carried out by the Marine Directorate (PWC) using a 1.8 m and a 14 m flume at the National Research Council between June and September 1985. This report describes the tests conducted, the results obtained, conclusions and recommendations.

The initial investigations into the entrance channel sedimentation problems were conducted by W.F. Baird and Associates (WFBA). In their report (submitted March 1984), entitled Engineering Investigations - Harbour Entrance, Stephenville, Newfoundland [1], it was recommended to repair the damaged head on the existing (north) breakwater, to construct a breakwater on the south side of the entrance channel so as to prevent the deposition in the entrance channel of sand and gravel from the south beach, and to dredge and re-align the navigation channel in order to reduce navigational hazard. Reference should be made to [1] for background information.

In this report, final designs for the north breakwater head reconstruction and the proposed south breakwater are presented. The north breakwater is presently under reconstruction and it is herein recommended to undertake a cost evaluation for the various layouts of the south breakwater.

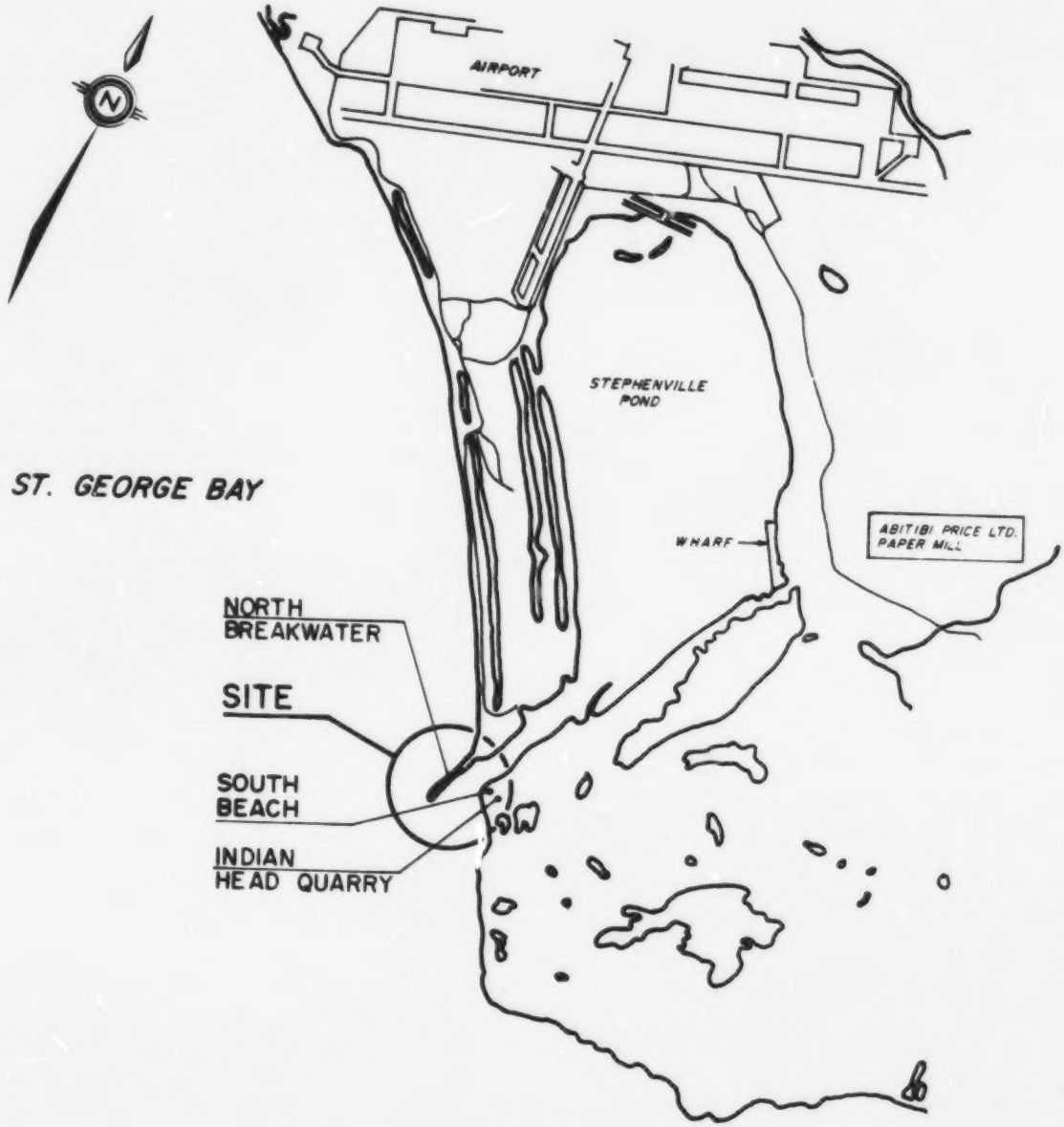


KEY PLAN

STEPHENVILLE, NFLD.

50 0 50 100 150 200 km

FIGURE 1.1



LOCATION PLAN
STEPHENVILLE, NFLD.

FIGURE 1.2

II ENVIRONMENTAL DATA FOR DESIGN

In this chapter, background information on the wave climate and water level fluctuations is summarized. Subsequently, two "design storm" profiles are developed for the model breakwater tests.

2.1 Wave Climate Study Conducted by WFBA.

The WFBA report [1] presented a detailed discussion of their study into the wind and wave climates at Stephenville, Newfoundland. They:

i) identified major storms in 1954-1955 which may have damaged the north breakwater while under construction. The information was obtained from records of hurricane tracks during those years and a wave hindcast which was undertaken for the period 1953 through 1955;

ii) made an extensive comparison between the wind fields recorded from Grindstone Island (1953-1982) and Stephenville airport (1953-1982). They concluded the Grindstone Island wind field was more complete and reliable;

iii) hindcasted 10 years (January 1, 1973 to December 31, 1982) of a deepwater wave climate for Stephenville using the Grindstone Island wind data base;

iv) obtained archived recorded wave data from the Marine Environmental Data Service (MEDS) for Stephenville (MEDS Station #20, October 7, 1974 to November 26, 1975);

v) performed a refraction analysis which provided a description of the wave climate at the north breakwater and the breaking wave climate halfway along the south beach;

vi) compared the (hindcasted) wave climate representative of the location of the MEDS Station #20 recorder with the recorded wave data for the same period of time. This confirmed the reliability of the hindcast and refraction - diffraction techniques used in [1];

vii) identified the major storms which occurred in the 10 year hindcast period and performed a statistical analysis to estimate the extreme value distribution of significant wave heights which describes the total population of storms. The storm distributions presented in [1] provided the following extreme value design guidelines for significant wave height (in 21 m of water in front of the north breakwater) as a function of return period;

H_s* (m)	Tp (s)	Direction	Return Period (years)
3.0	11	W to WSW	1
5.0	12	W to WSW	10
7.0	13	W to WSW	50

viii) determined the design high water level (large tides) to be 1.7 m and estimated the design storm surge to be 1.0 m.

2.2 "Design Storm" Profiles

The one in fifty year return storm having H_s = 7.0 m, T = 12 s** (in 21 m of water) and a design water level (DWL) = +2.5 m were adopted as design criteria for both the north and south model breakwater studies. After careful examination of the growth and duration of storm profiles at Stephenville from the MEDS recorded wave data, two storm profiles for the model breakwater tests were created by means of a stepped representation of the real growth

---

* The value given in [1] for H_s was actually a range of H_s. The values presented above are the upper limit of this range.

** Due to limitation in the synthesis of the wave climate, a 12 s wave period was used instead of 13 s.

curves. These storm profiles illustrated in Figures 2.1 and 2.2 were used as the "design storms" in the 2D and 3D model breakwater studies respectively. The storm profile is composed of many phases; i.e. an initial storm (stages 1 to 6) at the design water level (+2.5) whose return period is 1 in 50 years followed by an 18 hour duration (t_d) of the 10 year return significant wave at low water (stage 7A), and finally by another 18 hour duration (t_d) of the same wave at the design water level (stage 7B). If a section survived the design storm profile with a minimum level of damage, it was classified as stable. Generally, a small percentage (say 5%) of damage is considered acceptable [2], [3]. If a section survived the design storm profile, but showed excessive damage, remedial measures to reduce the percentage of damage were implemented and a new test was conducted. If a section failed suddenly during the course of the storm profile, the test was stopped.

Figure 2.3 illustrates the water level fluctuations of the stage 4 synthesized wave climate at the wave paddle, variance spectral density function of the synthesized wave, and the actual values of the synthesized wave parameters. The same plots for the remaining storm stages are presented in Appendix D.

STAGE	1	2	3	4	5	6	7A	7B
H_s (m)	3	4	5	6	7	6	5	5
T (sec)	8	9	10	11	12	11	10	10
t_d (hrs)	3	5	5	5	5	5	18	18

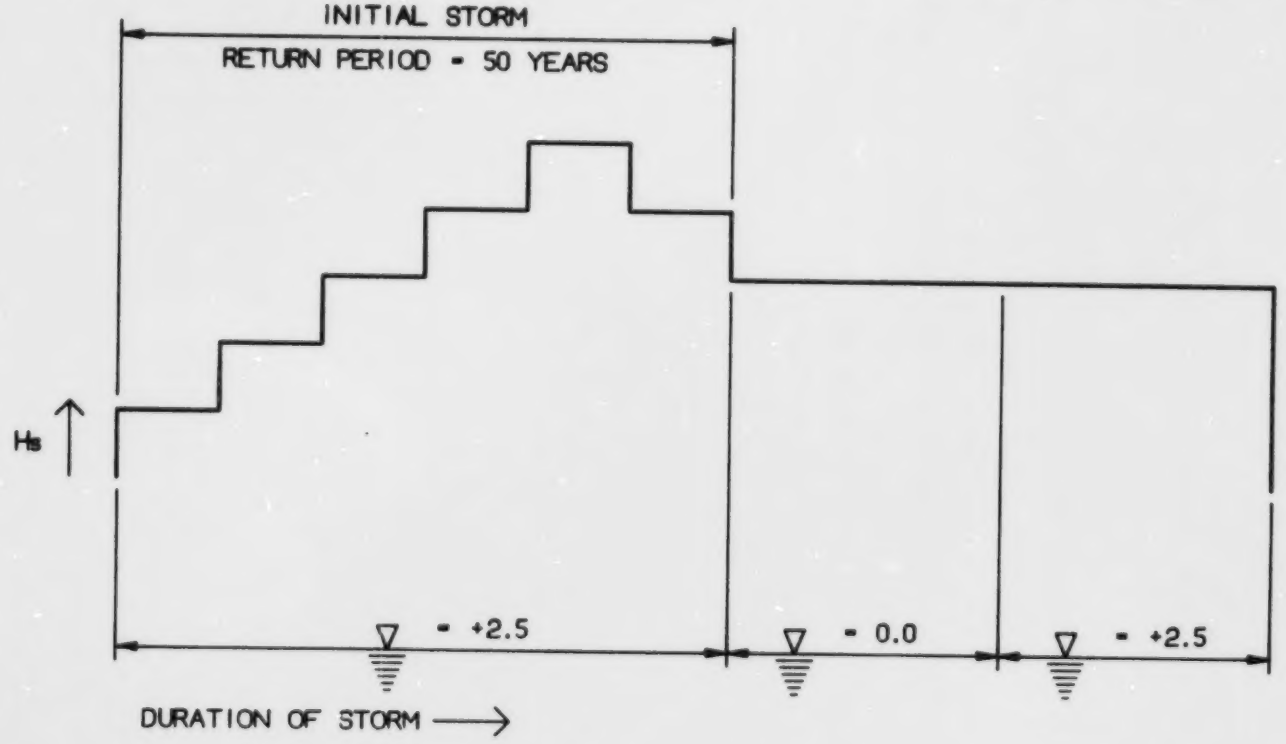


FIGURE 2.1

DESIGN STORM PROFILE FOR 2-D TESTS
(WAVE PARAMETERS ARE FOR 21 m OF WATER)

STAGE	1	2 (8)	3 (9)	4 (10)	5 (11)	6	7A	7B
H_s (m)	3	4	5	6	7	6	5	5
T (sec)	8	9	10	11	12	11	10	10
t_d (hrs)	5	7.5	7.5	7.5	5	5	18	18

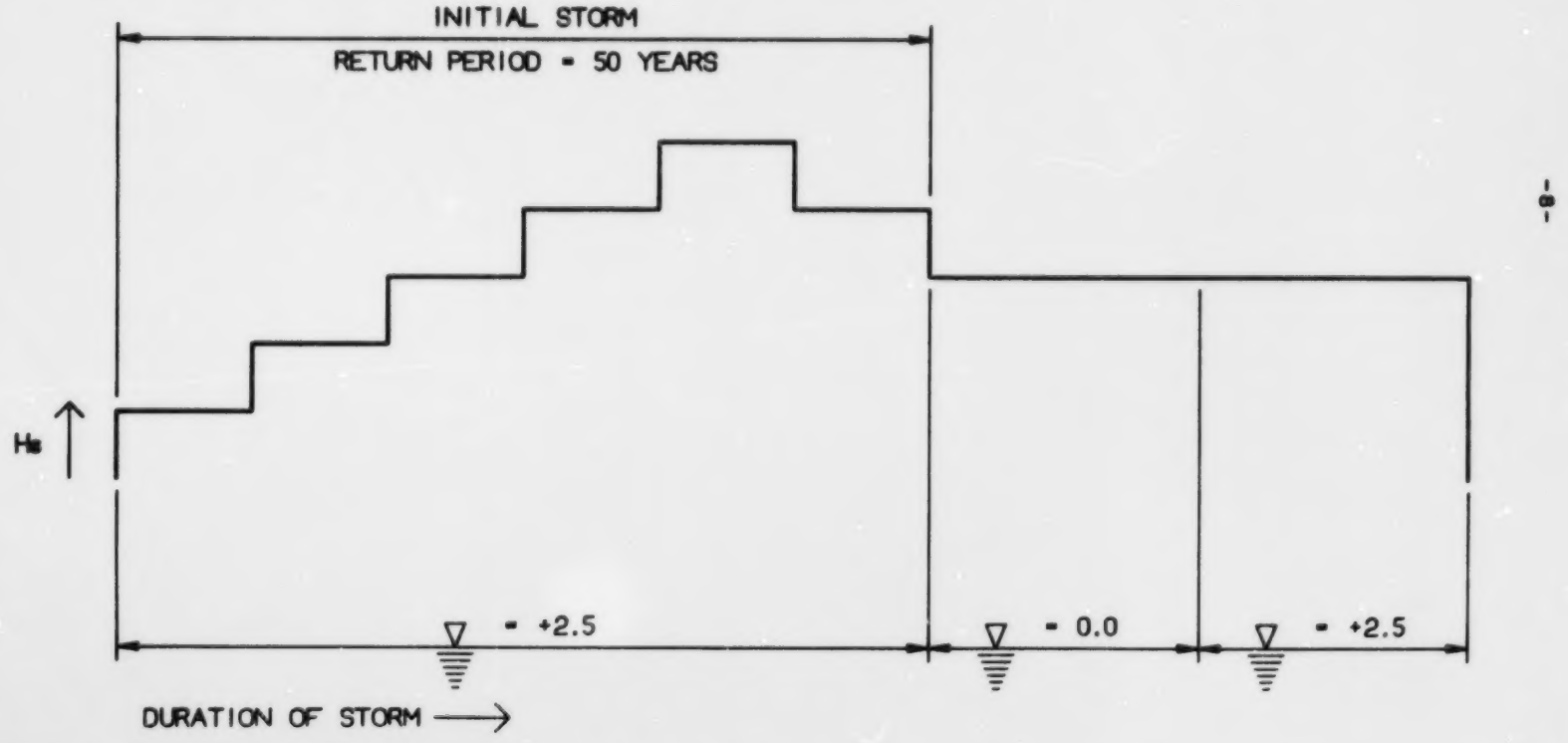
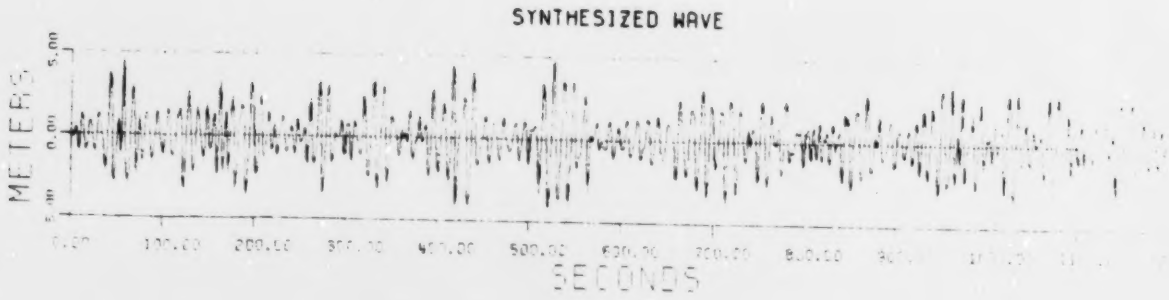


FIGURE 2.2

DESIGN STORM PROFILE FOR 3-D TESTS
(WAVE PARAMETERS ARE FOR 21 m OF WATER)

WAVE CHARACTERISTICS FOR STORM PROFILE STAGE 4



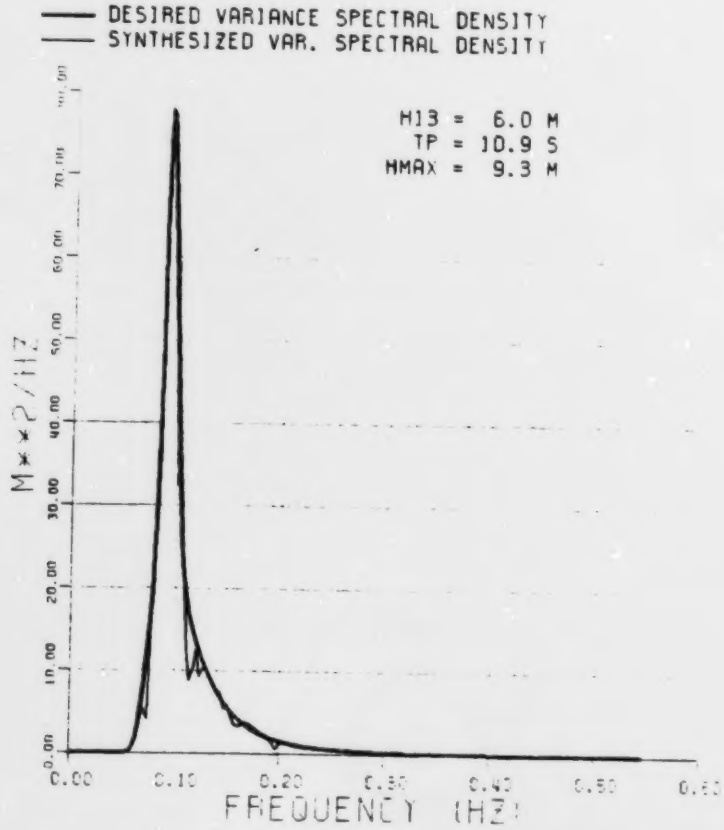


Figure 2.3

III NORTH BREAKWATER RECONSTRUCTION

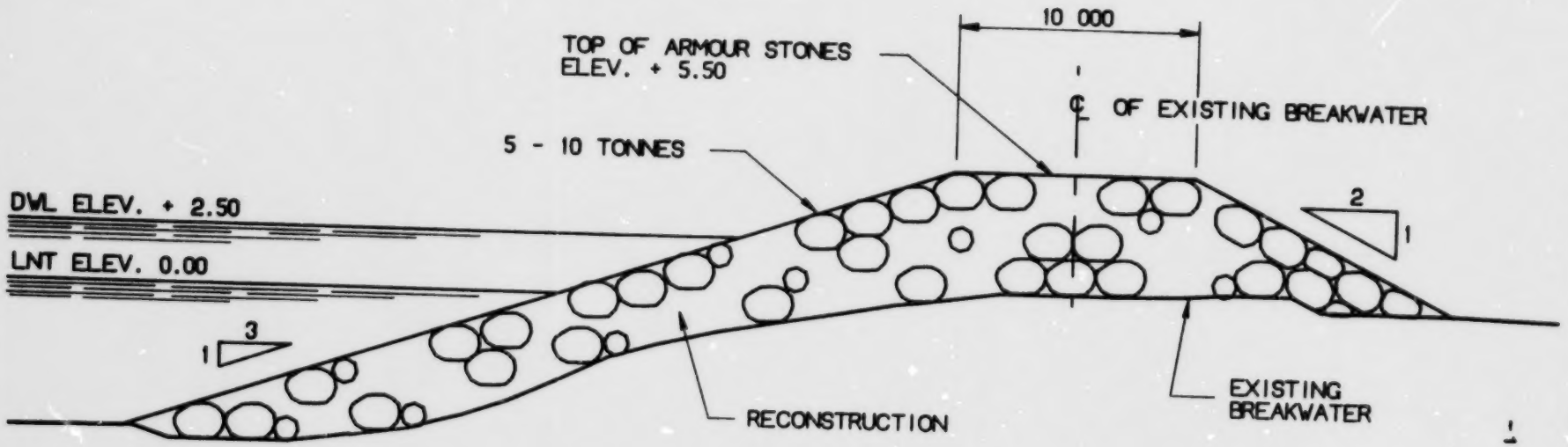
3.1 General

The north breakwater which is illustrated in Figure 1.2 was constructed in 1952-1953. Since initial construction, the head of the breakwater has been extensively damaged by storm action [1] and the overall length of the breakwater has been reduced by more than 100 m. Three localized areas along the trunk on the north side of the breakwater have also received small measures of damage. It is evident that the head of the north breakwater is particularly vulnerable to wave damage. The wave climate at the breakwater and storm analysis presented in [1] substantiate this claim. That is, the direction of wave attack at the breakwater during major storms is consistently from the WSW which constitutes a direct wave attack on the breakwater head. In this chapter, some details and discussion of the 2D and 3D model breakwater tests conducted for the reconstruction of the north breakwater head are presented.

3.2 Two Dimensional Flume Tests: North Breakwater Head Reconstruction

Six 2D (1:30 scale) flume tests were conducted using the 1.8 m flume at the NRC. Both the existing structure (Section 2 + 665, Figure 3.1) with the proposed reconstruction were modelled. A summary of parameters used in the six 2D tests and comments are listed in Table 3.1. The one test made at Section 2 + 825 was intended to investigate the reconstruction in a greater depth of water.

Although the wave attack is directly on the head of the north breakwater (i.e. a 3D phenomenon) 2D flume tests were conducted to get an initial indication of armour stone stability on a given slope. A 2D model can be constructed and tested in one day and many such



EXISTING SECTION 2 + 665 (SEE FIG. 3.6 FOR LOCATION OF SECTION)

SCALE 1:250

NOTES :

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2D TEST SECTION OF EXISTING
BREAKWATER AND RECONSTRUCTION
TEST 1

FIGURE 3.1

TABLE 3.1
Summary of 2D Flume Tests

Test	Armour Stone (tonnes)	Front Slope	Back Slope	Crest Width	Existing Section	Deviation From Figure 2.1	Comments
1	5-10	3:1	2:1	10 m	2 + 665	Stage 7 H_s = 6.0 m T = 11 sec. SWL = 0.0 td = 12 hrs.	- very stable front slope - minor damage - 10 m crest width is insufficient, damage at back of crest.
2	5-10	2:1	2:1	12 m	2 + 665	Stage 6 was 3 hrs.	- front slope unstable - 40% damage - back of crest is stable
3	5-10	2.5:1	2:1	12 m	2 + 665		- front slope stable - back slope stable - crest is stable
4	2-10	2.5:1	2:1	12 m	2 + 665		- front slope unstable - 35% damage
5	2-10	3:1	2:1	12 m	2 + 665		- front slope stable - back slope stable - crest stable
6	2-10	3:1	2:1	12 m	2 + 825	Stage 7 was not performed.	- same as test 5

- All crest elevations = + 5.5 m
- % damage refers to the percentage of displaced stones from the uppermost layer of armour stone layers.

tests can therefore be performed over a short period of time. On the other hand, a 3D model requires several days for the construction and testing cycle. In other words, the 2D flume was utilized to obtain a first approximation of the stone size and slope requirements.

Reflection in 2D flume testing can sometimes cause undesirable and unrealistic wave characteristics (i.e. standing wave patterns). Although the 2D flume at the NRC is specially equipped to minimize wave reflection, a reflection array was used to measure the actual values of the reflected wave heights and the reflection coefficients. Throughout all of the tests, the reflection coefficient varied between 3.5% and 8% which is deemed quite acceptable.

3.2.1 Conclusions from 2D Tests

i) It became apparent from test #1 that a 10 m crest width was insufficient. As the larger waves in the storm hit the structure, overtopping occurred. This overtopping exerted drag forces on the stones at the back of the crest which subsequently displaced them. Following tests utilized a 12 m crest width which proved stable in all the tests.

ii) When utilizing a 2^t-10^t armour stone, a 3:1 slope is required for stability under the given 1:50 year return storm.

iii) When utilizing a 5^t-10^t armour stone, a 2.5:1 slope is required for stability under the given 1:50 year return storm.

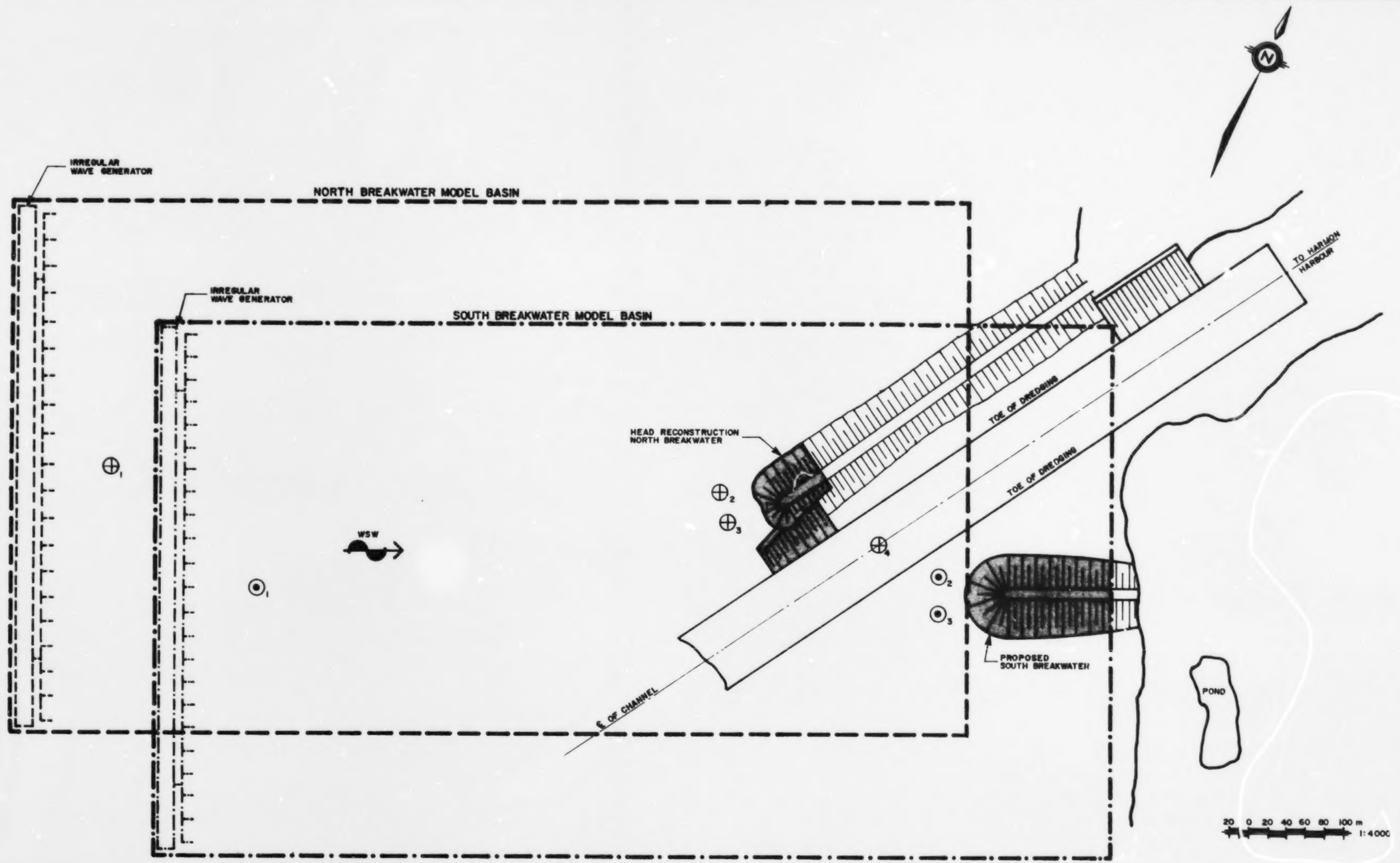
The conclusions listed above were used as initial input for the 3D studies discussed in the next section.

3.3 Three Dimensional Model Tests - North Breakwater Head Reconstruction

3.3.1 General

In any model breakwater study where the stability of the breakwater head is in question, 3D model studies are required to correctly represent the phenomenon. This is particularly applicable to Stephenville where the storm waves attack the head of the north breakwater directly.

Seven 3D model breakwater tests were conducted using the outdoor 14 m coastal flume at the NRC. The studies focused on repairs to the north breakwater head. The existing bathymetry (including the existing breakwater) and head reconstruction were modelled at a 1:40 undistorted geometric scale. The north breakwater model basin and head reconstruction (hatched area) are illustrated in Figure 3.2. The skin of the model bathymetry consisted of a pea-stone material which allowed for dredging of the entrance channel and other modifications (discussed later). The pea-stone material also ensured a rough turbulent boundary layer [15] thus reducing scale effects arising from the improper modelling of skin friction (i.e. a smooth surface). The direction of storm attack was from the WSW and the irregular wave generator was positioned at the far end of the basin in 21 m of (prototype) water. As previously discussed, the storm profile used in the 3D tests is illustrated in Figure 2.2. Although the maximum significant wave height for the 50 year return storm is 7.0 m, the irregular wave generator in the wave basin could only produce a maximum significant wave height in the order of 6.4 m in 21 m of water. This is not considered a major factor since even at this wave height, the waves were already breaking either on the structure, or offshore. Therefore, if the wave generator could have produced a 7.0 m significant wave, the waves would only have broken further offshore.



NORTH & SOUTH BREAKWATER – MODEL BASINS

FIGURE 3.2

The reconstructed model breakwater heads for test 6 and test 7 are shown in Figures 3.3 and 3.4 respectively. As can be seen from the pictures, the heads were sprayed with bands of coloured paint to facilitate observation of stone "displacement" (stone rolls into new stable location), "damage" (stone is completely removed from location) and overall mixing of armour stones.

3.3.2 Test Results

A point form summary of the 3D test parameters and comments on the north breakwater head reconstruction models are shown in Table 3.2. The relevant observations from the 3D tests conducted are as follows:

i) Test 1 - the breakwater head received excessive damage (approximately 20%). Majority of damage occurred at the H_s = 5 m level (stage 3) of the storm profile. The damage may be seen in Figure 3.5. Observe bulk of stone displacement occurred on the front face of the head and a localized region of damage on the south side of the head. This localized region will herein be referred to as the "damage area" (see Figure 3.6). The large percentage of stone displacement on the front face indicates the 5^t-10^t armour stone is not stable at a 3:1 slope, contrary to the conclusions arrived at in the 2D model study (Section 3.2.1 - iii).

ii) Test 2 - the armour stone on the breakwater head was increased to 7^t-10^t with a 3:1 slope. Although the damage was much reduced (approximately 5%), there was still some localized damage at the "damage area" (Figure 3.6).

iii) Test 3 - was a repeat of test 2 with the water level set to the LNT = 0.0 for the entire duration of the storm profile. Due to some scour at the base of the breakwater head, some sliding of stones from the south side of the breakwater head into the entrance channel occurred.

iv) Test 4 - the problem associated with test 3 was remedied by placing a 2^t-7^t stone apron which lined the side slope of the entrance channel up to the base of the reconstructed breakwater head. No sliding occurred during test 4, particularly during the low water level of stage 7. More will be said about this in the next section.

v) Test 5 - 8^t-12^t armour stone was used on the head in an attempt to find a no-damage section (i.e. stabilize the "damage area"). The results of this test were quite similar to those of test 2. That is, the damage was approximately 5% and was still localized to the "damage area".

vi) Test 6 - it was suggested by personnel at the NRC[1] that perhaps by increasing the slope to 2.5:1 and using 8^t-12^t armour stone on the head, better interlocking properties between the armour stones might be achieved and that perhaps the "damage area" might stabilize. This did not prove to be the case as a sudden failure on the front face of the breakwater head occurred. This indicates that at least, a 3:1 slope on the breakwater head is essential when utilizing armour stones up to 12 tonnes.

---

1 Joe Ploeg - Head of Hydraulics Laboratory, Mechanical Engineering Division, NRC - Private Communication.

TABLE 3.2
Summary of 3D Test Results (North Breakwater Head Reconstruction)

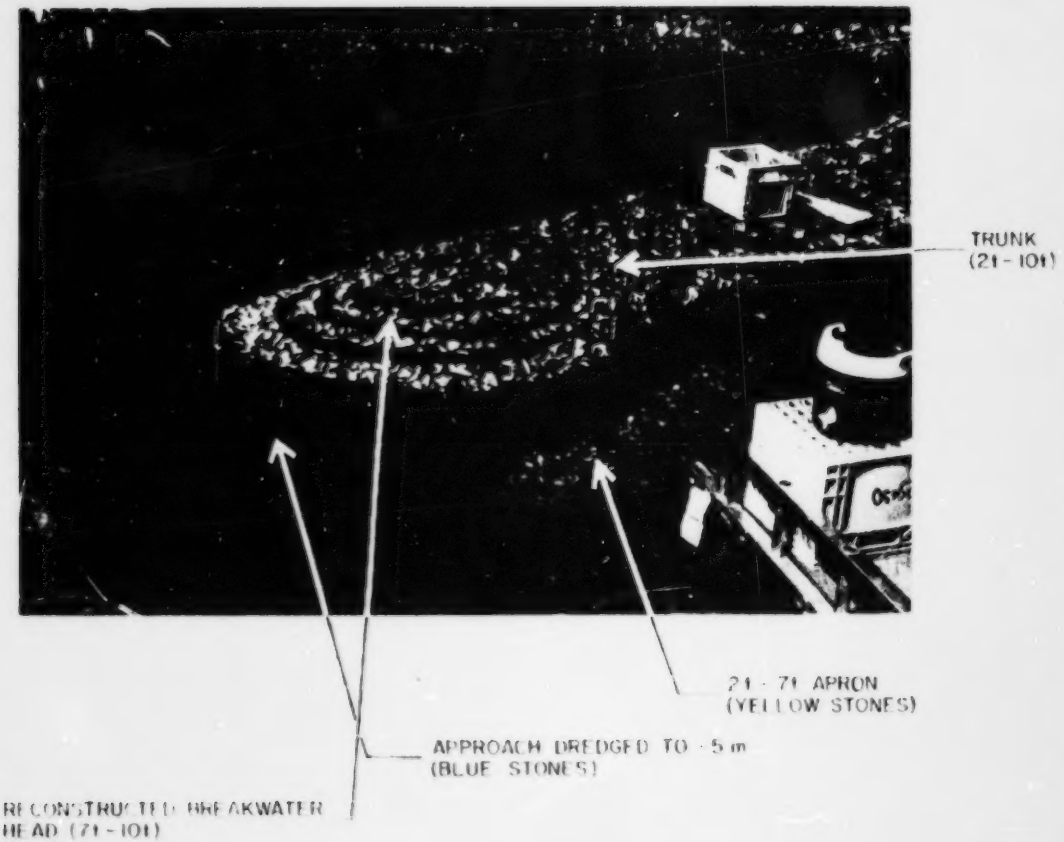
Test	Head Armour	Head Slope	Apron Armour	Apron Slope	Deviation from Storm Profile	Modifications	Observations and Comments
1	5^t-10^t	3:1	0		- test stopped after stage 5		- 20% damage on front face of head at completion of test - 5% damage in "damage area" - trunk stable
2	7^t-10^t	3:1	0		- test stopped after stage 5		- minor rocking, minor damage (approx. 5%) to front face of head and "damage area" - minor displacement - trunk stable
3	7^t-10^t	3:1	0		- test stopped after stage 5	Water level = 0.0 for entire test	- minor damage due to sliding of stones into entrance channel - 5% damage in "damage area" - trunk stable
4	7^t-10^t	3:1	2^t-7^t	3:1		Place apron on south side of breakwater head	- no sliding of stones into entrance channel - apron very stable, i.e. one stone displaced from apron - less than 5% damage in "damage area" - trunk stable
5	8^t-12^t	3:1	2^t-7^t	3:1	- test stopped after stage 5		- damage on head similar to test 2 - trunk stable
6	8^t-12^t	2.5:1	2^t-7^t	3:1	- test stopped after stage 5		- sudden failure of head after stage 3 - 30% damage - trunk stable
7	7^t-10^t	3:1	2^t-7^t	3:1	- test stopped after stage 5	Approach to head dredged to -5 m	- minimal damage (3 stones) to "damage area", almost "no damage" section - most acceptable test - breaking pattern of waves changed significantly - coloured contour indicators remain quite undisturbed - trunk stable

NOTES: Trunk Armour = 2^t-10^t (3:1) for all tests.
Damage defined as % of displaced stones from uppermost layer of armour stone layers. Since wave damage occurred on growth of storm profile, tests were stopped after stage 5. Stages 6 & 7 were used on test 4 to verify stability of section.

APRON
RECONSTRUCTED BREAKWATER
HEAD (81 - 121 ARMOUR, 2.5 : 1 SLOPE)
REMAINS OF EXISTING (DAMAGED)
BREAKWATER HEAD (YELLOW STONES)

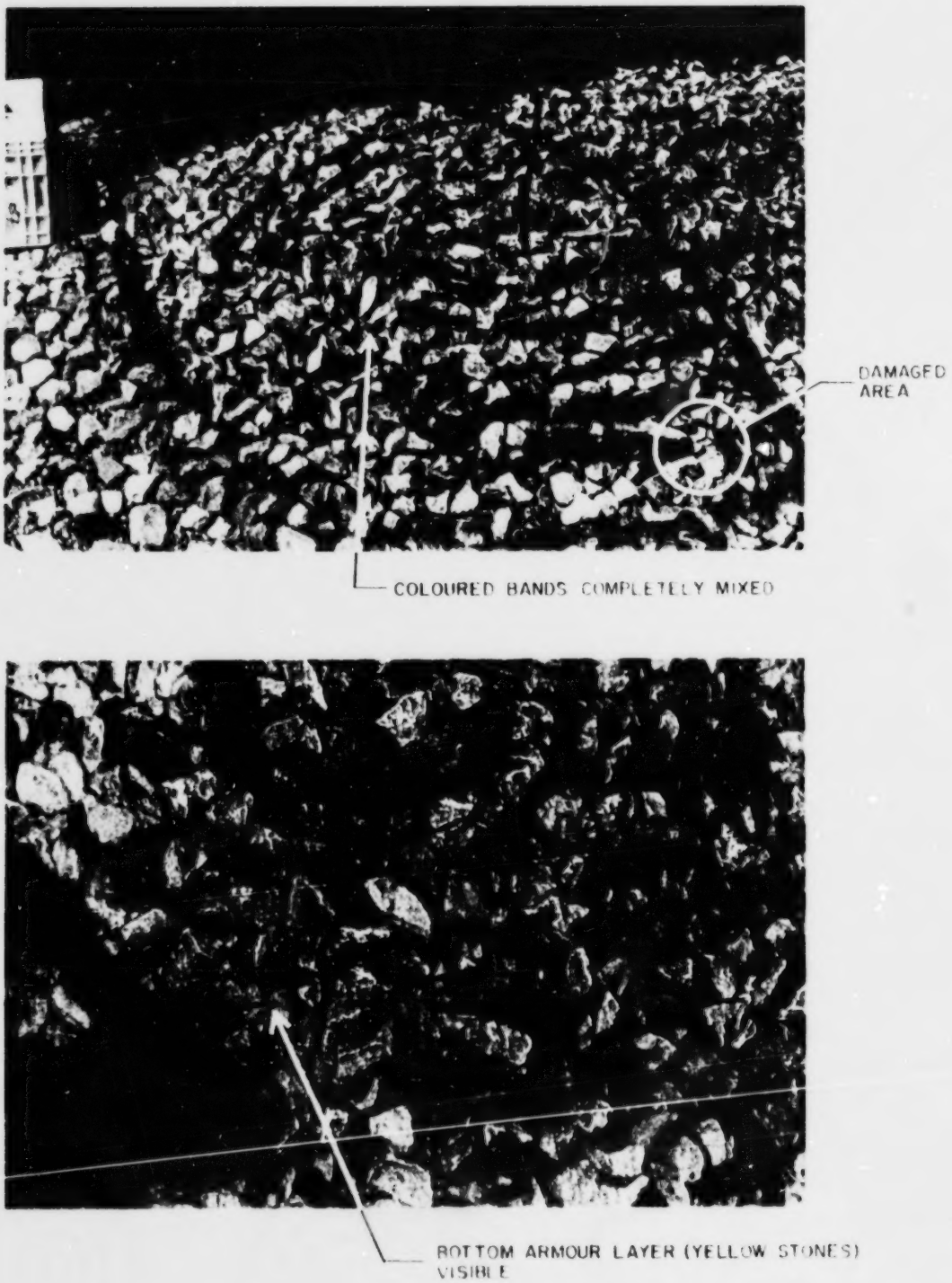
STEPHENVILLE NORTH BREAKWATER - BEFORE TEST 6

FIGURE 3.3



STEPHENVILLE NORTH BREAKWATER - BEFORE TEST 7

FIGURE 3.4



STEPHENVILLE NORTH BREAKWATER - AFTER TEST I

FIGURE 3.5

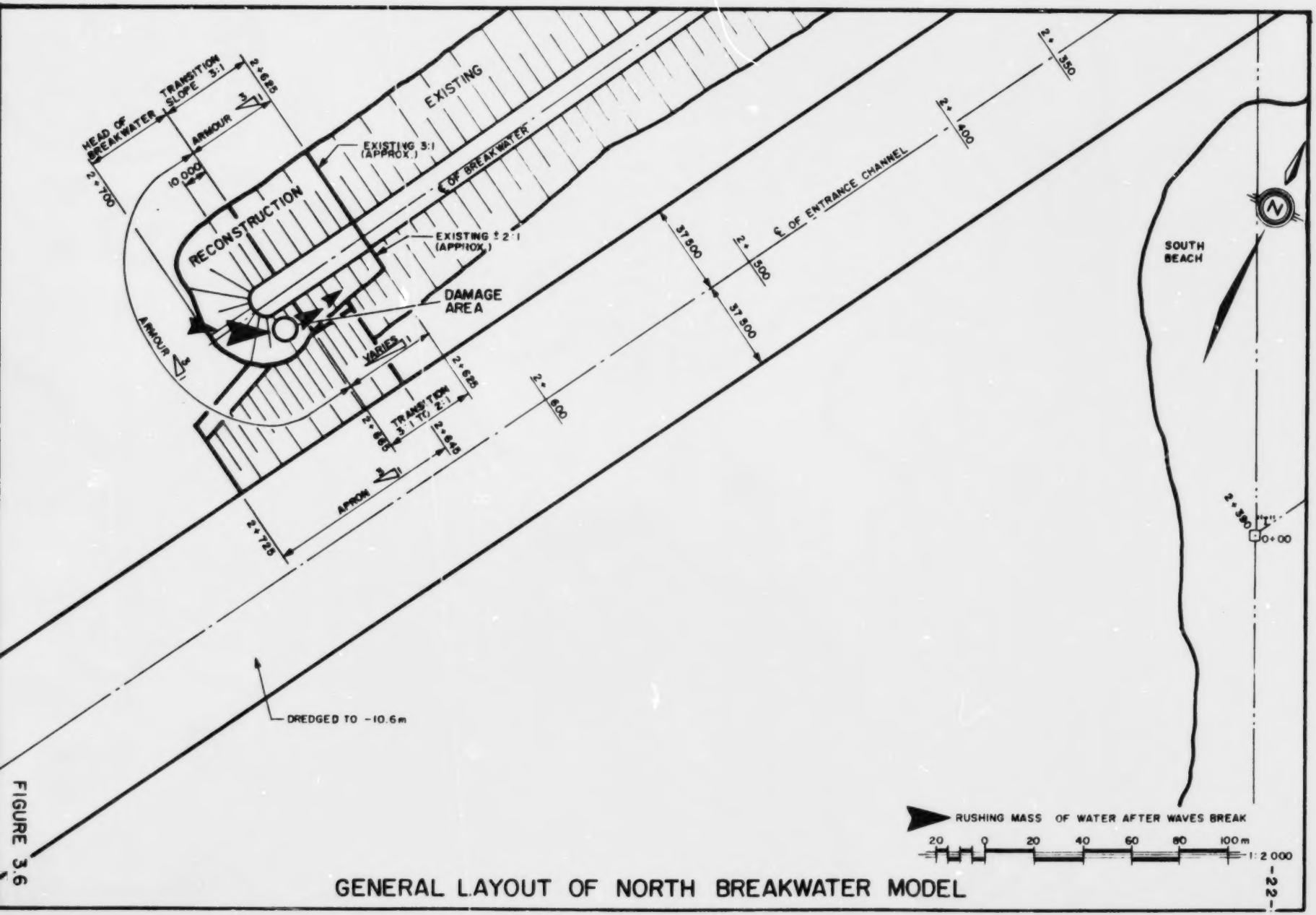


FIGURE 3.6

GENERAL LAYOUT OF NORTH BREAKWATER MODEL

vii) Test 7 - in this test it was decided to dredge the approach to the reconstructed head to -5 m in order to alter the breaking wave pattern of the incoming waves. This yielded very favourable results. The "damage area" only had three stones displaced and it was estimated that the overall damage was less than 2%. In fact, the painted lines on the breakwater head were significantly less mixed than all previous tests indicating a more overall stable structure.

A summary of the recorded wave characteristics from the above tests is presented in Appendix A.

3.3.3 Discussion of Tests

From the 3D tests conducted, it is clear that the parameter used in test 4 provided satisfactory results. These parameters were subsequently used to design the final sections for reconstruction purposes. The final design layout of the structure and relevant sections are shown in Figures 3.7 and 3.8 respectively.

Some discussion of the breaking wave pattern is useful at this point. It was previously mentioned that the original breakwater head was destroyed by storm action and the overall length of the breakwater was reduced by more than 100 m. As the waves approach the breakwater head, an obvious shoaling process occurs as the waves travel over the remains of the former structure. This shoaling affects the breaking pattern of the waves in that it causes a concentration of wave energy on the breakwater head. This can be observed from the tables in Appendix A where the recorded wave heights at gauge 3 (Figure 3.2) are consistently larger than those recorded at gauge 2.

This observation was confirmed in test 7 where the approach to the breakwater head was dredged to -5 m. It was quite apparent during this test that the breaking wave pattern was different from those of the previous tests. The dredging caused the breaking wave energy to be more spread out thus less intense than in the undredged breaking wave pattern. Also, the frequency of breaking waves on the structure was lower than previous tests. This leads to the conclusion that dredging the shoal in front of the reconstructed head to -5 m improves overall stability of the reconstructed breakwater head. However, due to the satisfactory results of test 4, this element was not included in the final design. The above conclusion is presented to help understand the breaking phenomenon. If a "no damage" structure is desirable, dredging of the approach to -5 m can be implemented anytime in the future.

It was also observed that after the waves break in front of the structure, a mass of water propagates ("rushes") forward. This mass of water then "rushes" around the breakwater head and along the south trunk (see arrows in Figure 3.6) resulting in large drag forces on the armour units on the south side of the head, i.e. the "damage area". It is these drag forces (rather than breaking wave energy) which consistently displaced armour stones from the "damage area".
This phenomenon of drag forces acting on armour units leeward of the breakwater head have been presented by others ([5], [6], [10],...) in the literature.

Finally, it was observed from test 3 that during periods of low water, the breaking waves and resulting drag forces caused some scour at the base of the breakwater head (on the south side). Consequently, some sliding of armour units from the reconstructed head onto the side slopes of the entrance channel occurred. This problem was remedied in subsequent tests by the placement of the 2^t-7^t stone apron shown in Figures 3.7 and 3.8A.

Throughout all of the tests, the trunk of the breakwater reconstruction (2^t-10^t armour, 3:1 slope) was stable. In fact, the trunk which was never rebuilt, survived the 7 tests conducted.

3.4 Channel Re-Alignment and Dredging

Due to the deposition in the entrance channel of armour stones and core material from the damaged head of the north breakwater, vessels entering the entrance channel must negotiate a 4° bend (in the channel alignment) at low speeds. This creates a navigational hazard. A straight 75 m wide (dredged to -10.6 m) entrance channel was also modelled (see Figure 3.7) in the 3D tests discussed above.

3.5 Conclusion

The final design for the north breakwater head reconstruction (Figures 3.7 and 3.8) was opened for tender in August 1985 and was subsequently awarded to C.A. Pitts for $2,485,860. Included in this contract was a requirement to dredge and clear the entrance channel as discussed in Section 3.4

The north breakwater head is presently under reconstruction.

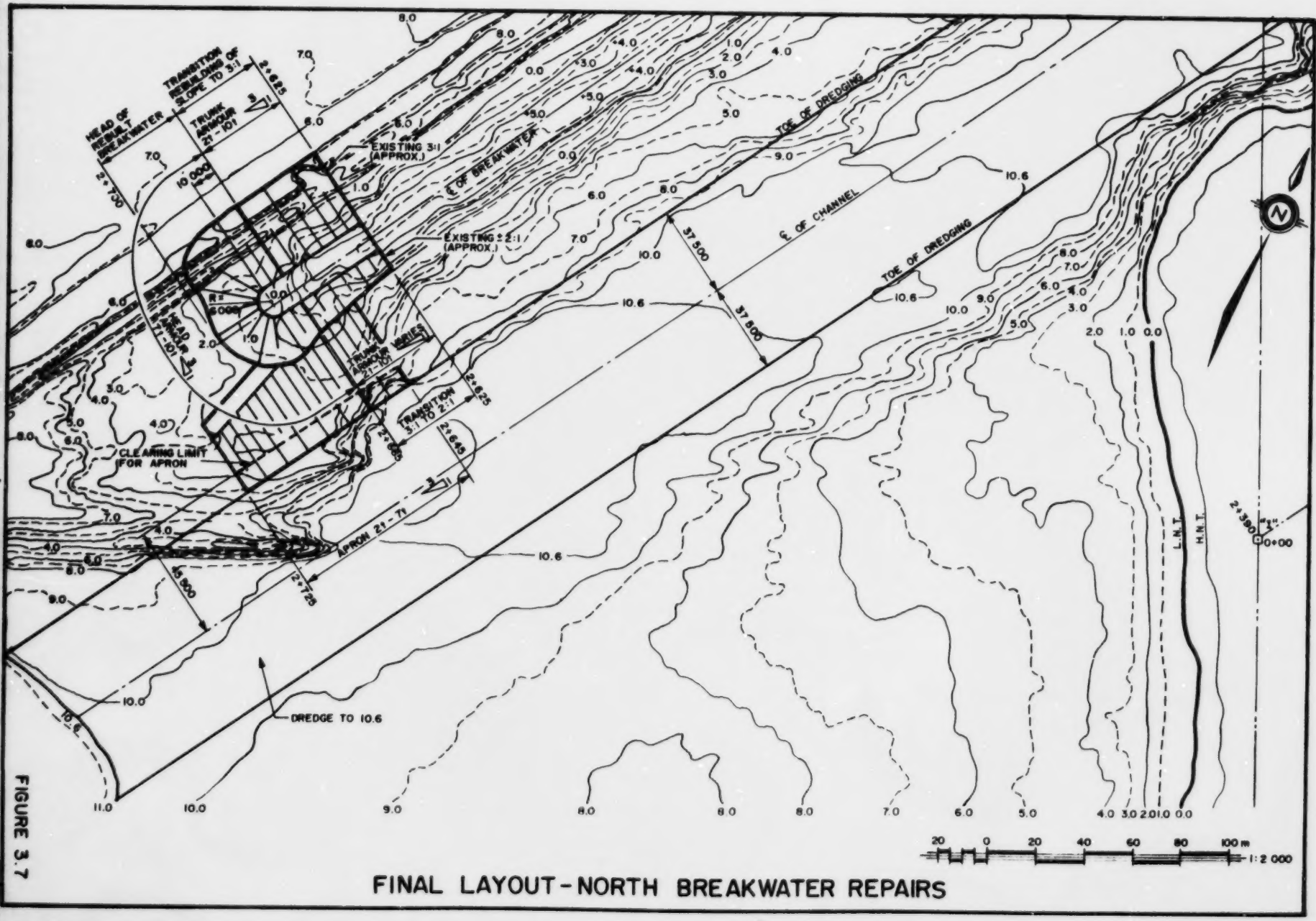
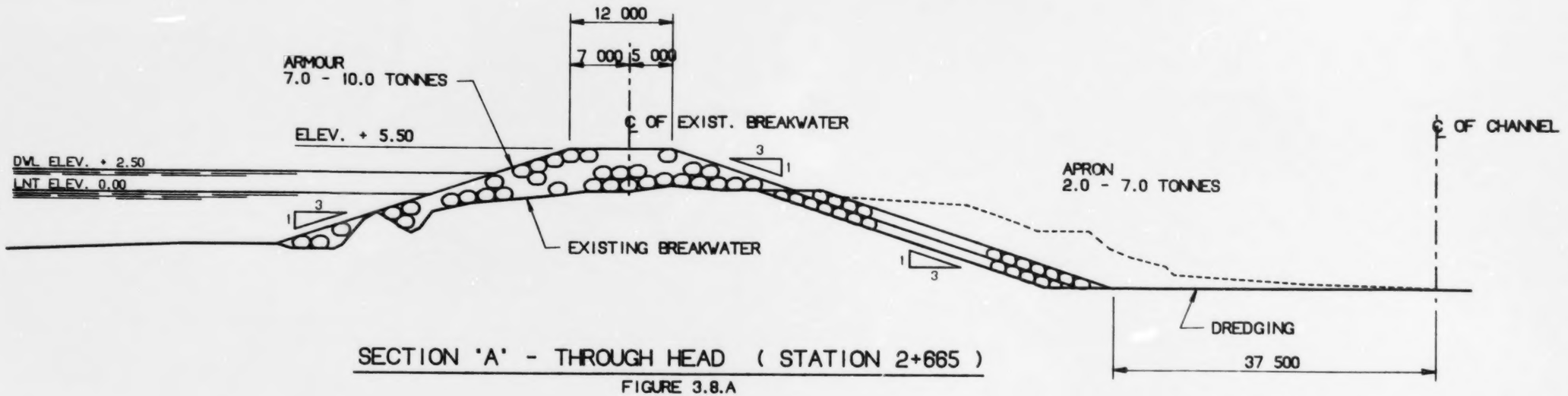


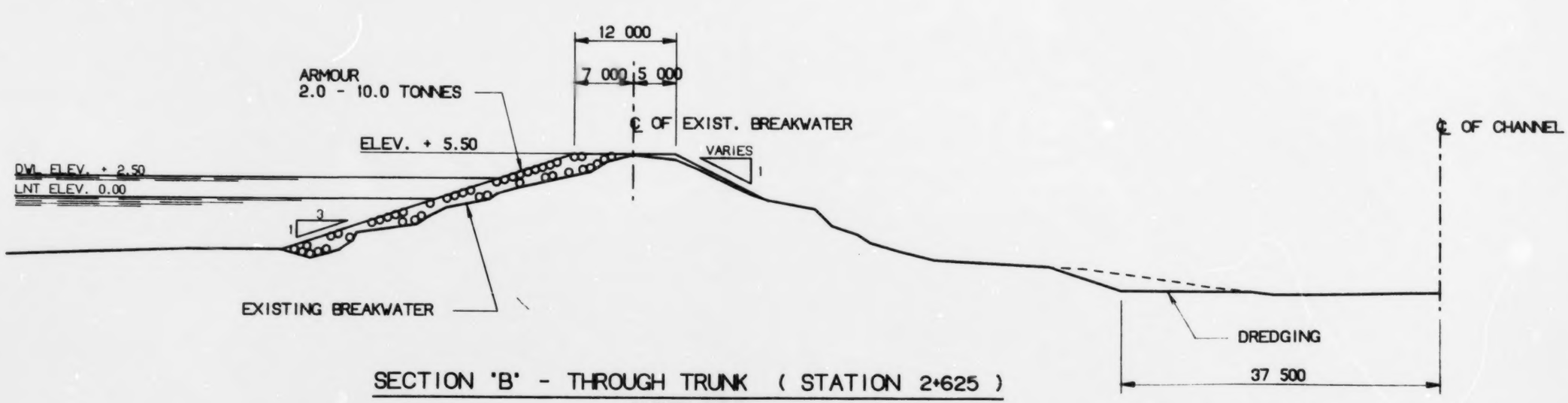
FIGURE 3.7

FINAL LAYOUT - NORTH BREAKWATER REPAIRS



SECTION 'A' - THROUGH HEAD (STATION 2+665)

FIGURE 3.8.A



SECTION 'B' - THROUGH TRUNK (STATION 2+625)

FIGURE 3.8.B

STEPHENVILLE, NFLD.

SECTIONS
NORTH BREAKWATER
HEAD RECONSTRUCTION

NOTE :

ALL DIMENSIONS IN MILLIMETERS

SCALE 1:500

FIGURES 3.8.A & 3.8.B

IV SOUTH BREAKWATER

4.1 General

In [1], WFBA studied the sedimentation patterns in the entrance channel and concluded that wave induced currents are established on the south beach which are capable of transporting coarse sand or gravel into the dredged entrance channel (Figure 4.1). It was therefore recommended (in [1]) to construct a breakwater on the south side of the entrance channel to prevent the deposition in the entrance channel of sand and gravel from the south beach. It was also concluded that sand and gravel on the south beach can be actively eroded by the action of waves beginning in a depth of 8 m. Therefore, WFBA recommended that the end of the breakwater extend to a depth of 8 m.

Six 3D breakwater model tests were conducted at an undistorted geometric scale of 1:40 in the outdoor coastal flume at the NRC (see south breakwater model basin Figure 3.2). The purpose of these tests was to investigate a suitable design for this breakwater. Of these tests, four tests were required to develop a stable conventional breakwater structure and the remaining two tests were to evaluate the berm design breakwater suitability.

4.2 Conventional Design

Four (conventional design) breakwater model tests were conducted for a breakwater located on the south beach as shown in Figure 4.2. Here again, the south breakwater models were sprayed with bands of coloured paint as was done for the north breakwater models. The design storm profile and direction of wave attack adopted for the south breakwater were the same as those adopted for the north breakwater 3D study as discussed in Section 3.3.1.

4.2.1 Summary of Test Results

Table 4.1 summarize the parameters used and observations from the four conventional breakwater model tests performed. Typical sections (crest width modified from 12 m to 5 m) are shown in Figures 4.9A and 4.9B. Note that the first two tests did not incorporate the north breakwater into the model basin, whereas, tests 3 and 4 did include the north breakwater to account for its protective effects. A short discussion on the relevant observations is presented below.

i) Test 1 - by the end of the design storm, the head of the breakwater received approximately 15% stone damage with the majority of this damage having occurred during the stages 4 and 5 of the storm profile. The coloured bands of paint on the front face of the head were completely mixed, i.e., large "displacement" of stones occurred.

At three locations on the head, the bottom armour layer was completely exposed, and at one of these locations, the filter layer was also visible. The trunk received moderate damage at three locations. On the north side of the trunk, one of these damaged locations also exposed the filter layer.

Overall, the results of test 1 were deemed UNSATISFACTORY.

ii) Test 2 - at the conclusion of test 2, here again, the head of the breakwater had received approximately 15% stone damage. The additional armour stone layer on the head caused eddy shedding at the intersection of the head and the trunk due to the sudden transition of geometry. Since there was no observable significant

damage to the trunk of the breakwater in this test, it was concluded that the added layer of armour stones on the head of the breakwater increased the stability of the trunk armour stones.

However, the results were still considered UNSATISFACTORY.

iii) Test 3 - the configuration of test 1 was repeated, however here, the north breakwater was introduced into the south breakwater model basin as shown in Figure 3.2. The results of this test were somewhat more favourable than tests 1 and 2. The north side of the breakwater head was reasonably stable. That is, some displacement and mixing of stones occurred, but no damage on the north side of the head was noted. Both sides of the trunk were also very stable, however, approximately 10% damage still occurred on the south side of the breakwater head. Results still considered UNSATISFACTORY.

iv) Test 4 - this test employed a heavier stone consisting of 10^t - 13.5^t on the breakwater head. This structure received the least amount of damage (<5%) and the least amount of stone mixing on the breakwater head. Observe from the "after test 4" photographs shown in Figure 4.3 that only a few locations had minor stone displacement. Based on these results, it was concluded that the conventional breakwater modelled in test 4 was a STABLE structure.

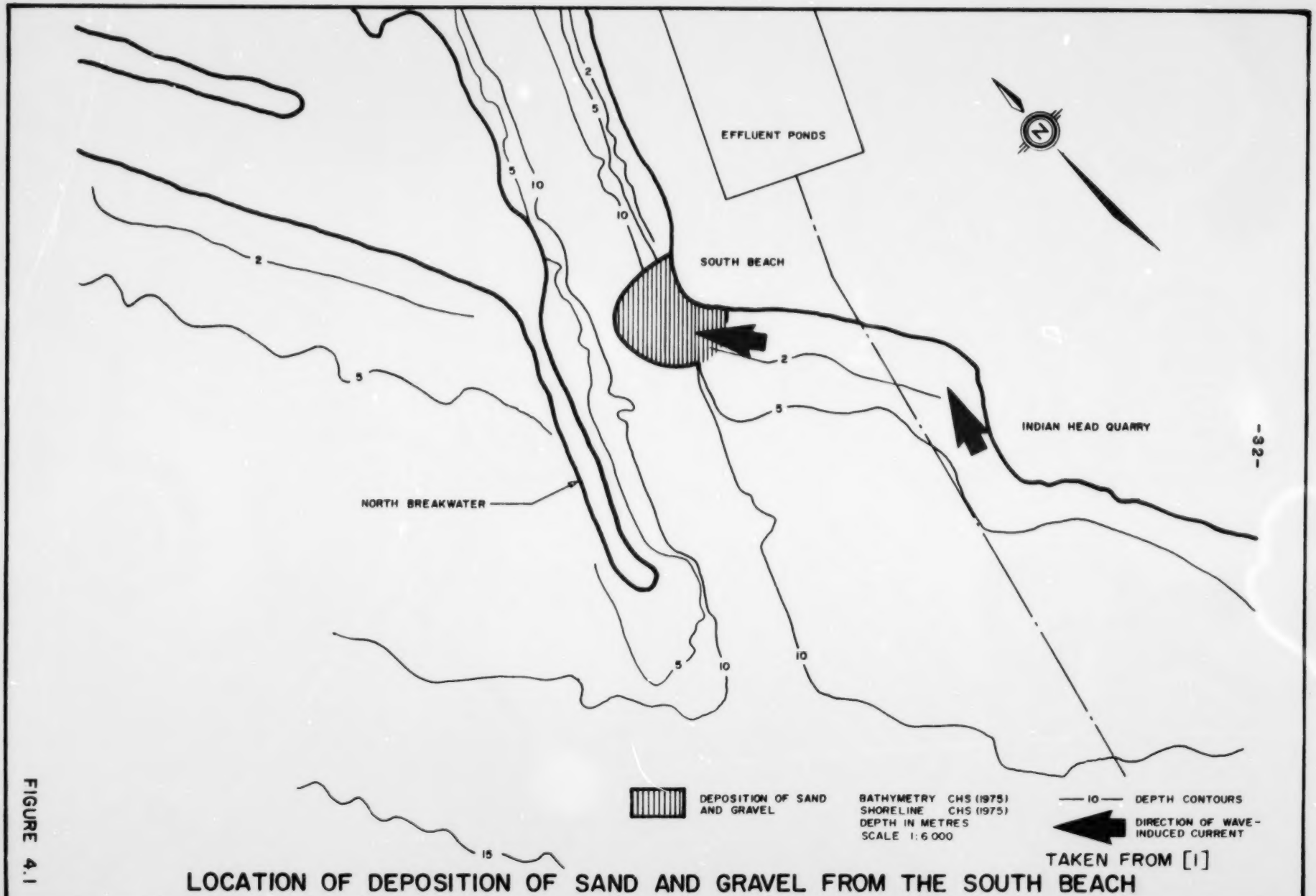
A summary of the recorded wave characteristics from the south breakwater (conventional structure) model tests is presented in Appendix B.

TABLE 4.1

Summary of Test Results - South Breakwater - Conventional Structure

Test	Armour on Head	No. of layers of armour stone	North breakwater incl. in model basin	Observations
1	7^t-10^t	2	No	- excessive damage to armour layer at several locations on head; - approximately 5% damage to trunk; - consistent damage to both sides of breakwater; - filter layer exposed at 2 locations; - overall unsatisfactory results; - much mixing of armour stone on head; - approximately 15% damage.
2	7^t-10^t	3	No	- excessive damage to no. 1 armour layer on head; - results considered unacceptable; - consistent damage on both sides of head; - much mixing of armour stones on head; - approximately 15% damage; - trunk very stable.
3	7^t-10^t	2	Yes	- north side of breakwater head held up well; - south side still received damage and mixing of armour stones; - not as severe as test 1 and test 2; - approximately 10% damage; - trunk very stable.
4	10^t-13.5^t	2	Yes	- acceptable levels of damage; - only a few stones displaced from head; - therefore declared a STABLE head; - approximately 5% damage; - trunk very stable.

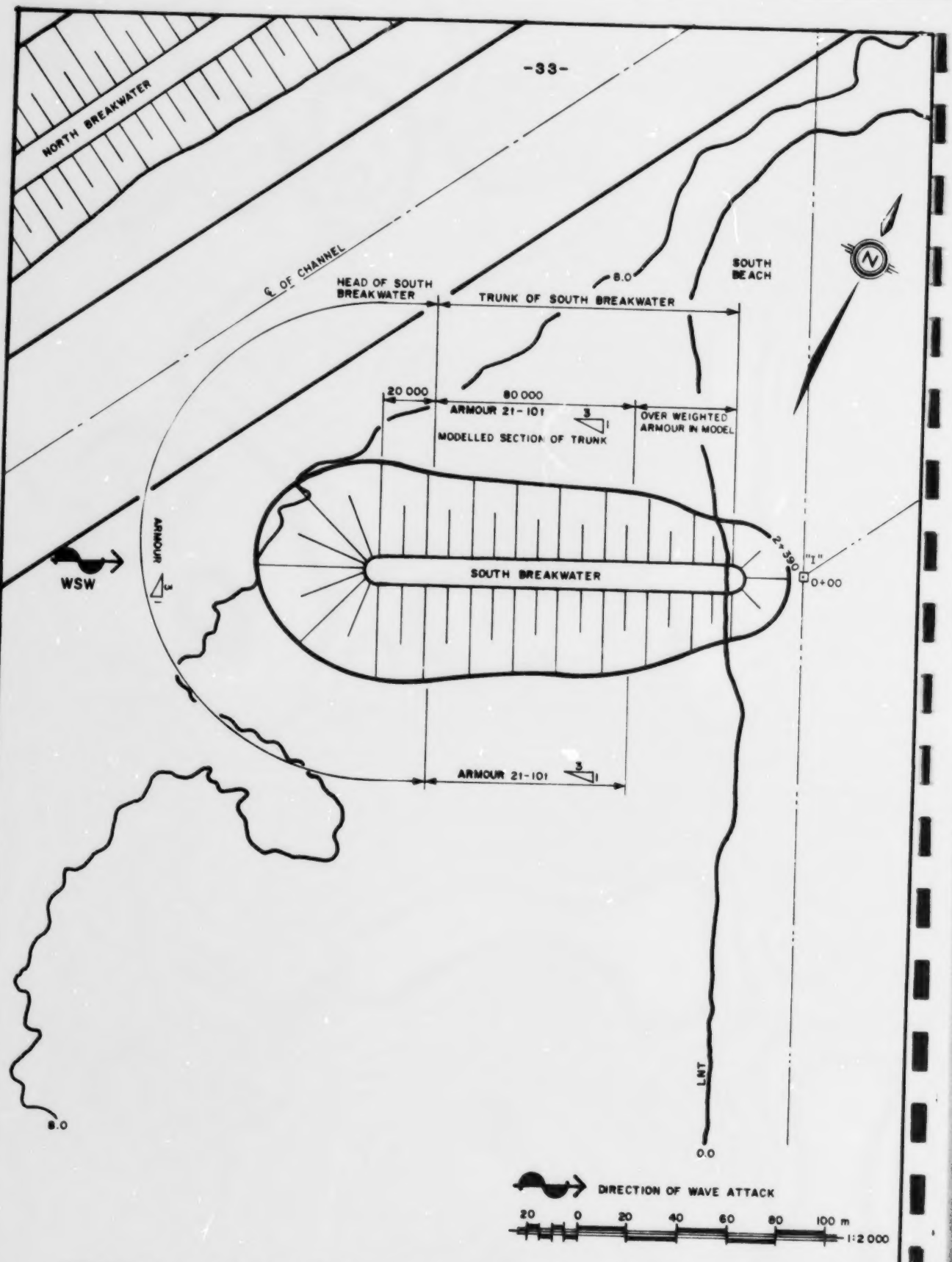
- Common parameters for the above tests:
  - Wave direction = o° (i.e. directly on central line of breakwater)
  - Side slopes on head = 3:1
  - Trunk armour = 2^t-10^t
  - Side slopes on trunk = 3:1
  - Crest elevation = +6.0
  - Crest width = 12 m
  - Filter layer = 0.5^t-1^t
- Damage defined as % of displaced stones from the uppermost layer of armour stone layers.
- Storm profile, Figure 2.2 stages 1 to 7A.



TAKEN FROM [1]

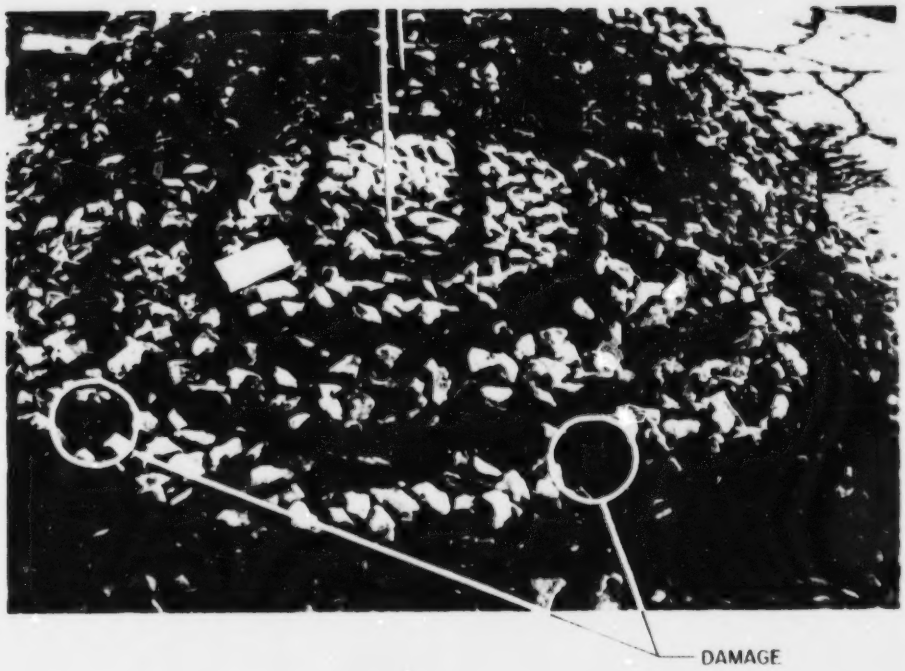
FIGURE 4.1

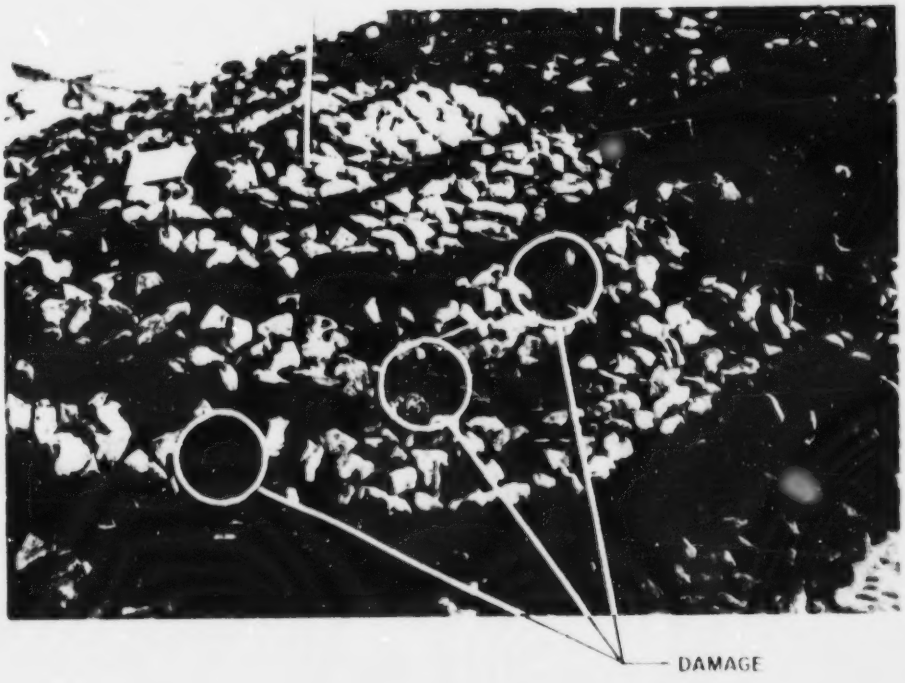
LOCATION OF DEPOSITION OF SAND AND GRAVEL FROM THE SOUTH BEACH



GENERAL LAYOUT OF SOUTH BREAKWATER MODEL TESTS
(CONVENTIONAL STRUCTURE)

FIGURE 4.2





STEPHENVILLE SOUTH BREAKWATER - AFTER TEST 4

FIGURE 4.3

4.2.2 Conclusions from Test Results

As in the case of the north breakwater study, the head of the breakwater experienced virtually the entirety of wave damage from storm action. One therefore concludes from these tests that the south breakwater head should be designed using 10^t - 13.5^t armour stone (3:1 slope) if a minimum level of damage (say 5%) is considered acceptable. The model study also indicates that 2^t-10^t armour stone (3:1 slope) is stable on the trunk of the breakwater.

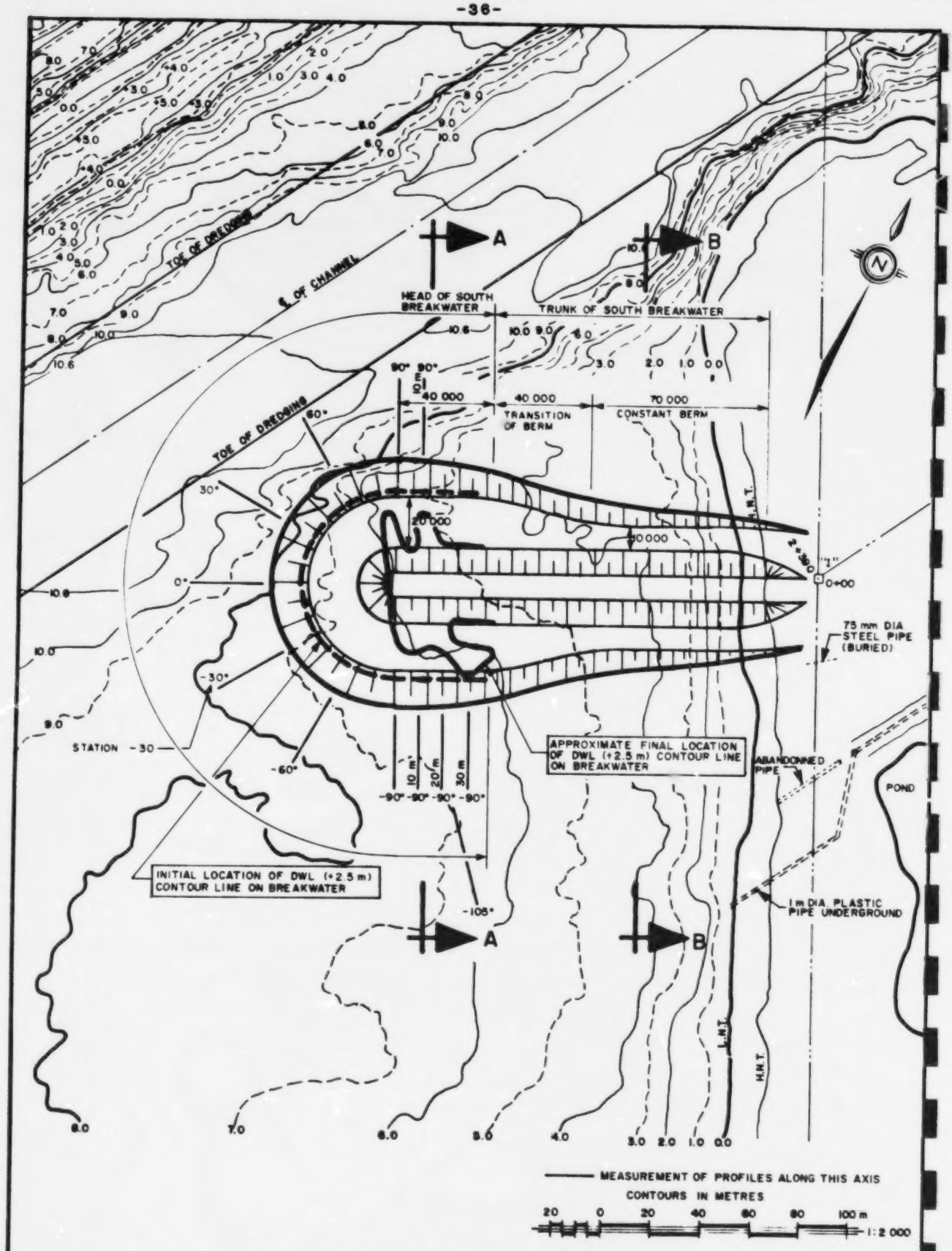
Since the head of the south breakwater is in a greater depth of water than that of the north breakwater, the wave climate which attacks the south breakwater is more severe (see Appendix A and B for gauges 2 and 3). It is essentially for this reason that the head of the south breakwater requires heavier armour stones than that of the north breakwater.

4.3 Berm Design

Two berm design breakwater model tests were performed in an attempt to provide an alternative design and perhaps a more cost effective structure for the south breakwater (the berm design breakwater concept is fully reviewed in [1] and discussed in [11]). The general arrangement and typical sections of test 2 are shown in Figures 4.4 and 4.5 respectively. Test 1 differed from test 2 only in that test 1 employed a 16 m width of berm over the entire length of the structure. Here too, the north breakwater was geometrically modelled in the south breakwater model basin.

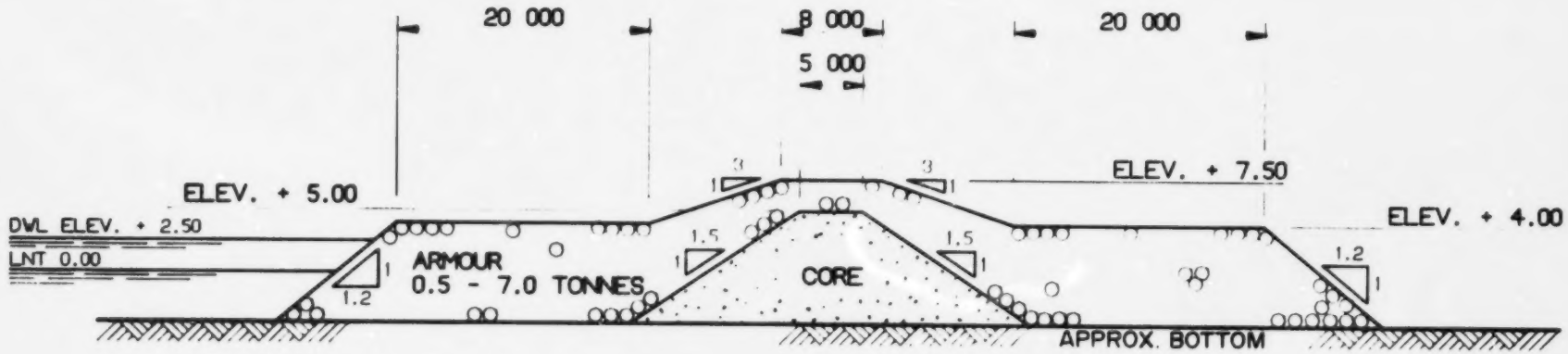
For both berm tests, the following common parameters were used:

Armour stone range	= 0.5^t - 7.0^t
Crest width	= 12 m
Crest elevation	= + 7.5 m
Berm elevation	= +4.0 m



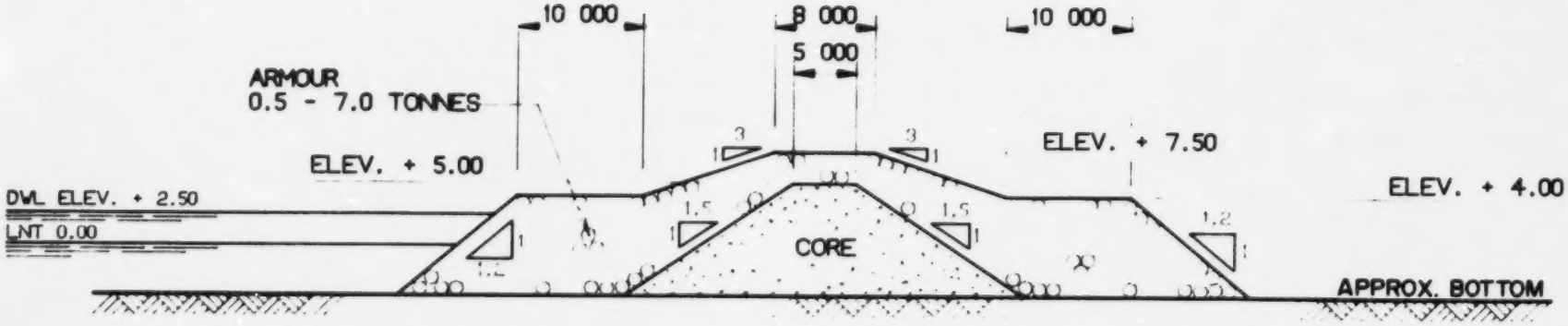
STEPHENVILLE SOUTH BREAKWATER
LAYOUT OF TESTED BERM STRUCTURE
(TEST 2)

FIGURE 4.4



SECTION THROUGH HEAD "A" (Berm 20 m)

FIGURE 4.5.A



SECTION THROUGH TRUNK "B" (Berm 10 m)

FIGURE 4.5.B

FIGURES 4.5.A & 4.5.B

STEPHENVILLE, NFLD.

SECTIONS THROUGH TESTED BERM DESIGN (TEST 2)

SCALE 1:500

NOTES :

ALL DIMENSIONS IN MILLIMETERS

The 0.5^t - 7.0^t armour stone was mixed in accordance with the stone gradation curve shown in Figure 4.6. The characteristics of the gradation curve are representative of the yields from typical quarry operations.

A key element behind the berm design concept is the proper development of the "equilibrium profile" or "S" shaped profile. That is, with wave action, the initial berm design cross-section (Figure 4.5) will develop in such a manner as to form an "equilibrium profile" (or "S" shaped profile) of variable slope. Once equilibrium has been reached, it is then desirable to have a minimum thickness of (say 3) armour stones from the periphery of the developed structure to the periphery of the core. If the profile development "eats" into the core, a breach in the structure will quickly occur. These findings have been reported in the literature ([10], [11], [1], [15],...). Therefore in a berm design breakwater model study, it is important to monitor the "breakwater" profile evolution and to ensure that the "breakwater" profile has fully developed at the conclusion of the test.

Considering this, the design "storm profile" adopted for the berm design tests consisted of stages 1 through 11 shown on Figure 2.2. The developing profiles were measured after stages 6, 7 and 10 at the stations shown in Figure 4.4. These profiled sections are presented in Appendix C and are further discussed in the next section.

4.3.1 Results of Berm Design Tests

Both tests 1 and 2 failed during stage 9 and 10 of the storm profile respectively and in quite a similar manner. The wave attack developed the front face of the breakwater head to such an extent that the core material was eventually exposed. Exposure of the core material first occurred along stations 0° and -30° (Figure 4.4). Once this occurred (late in stage 9 for test 1 and early in stage 10 for test 2) the rate of damage accelerated dramatically and the tests were terminated at the

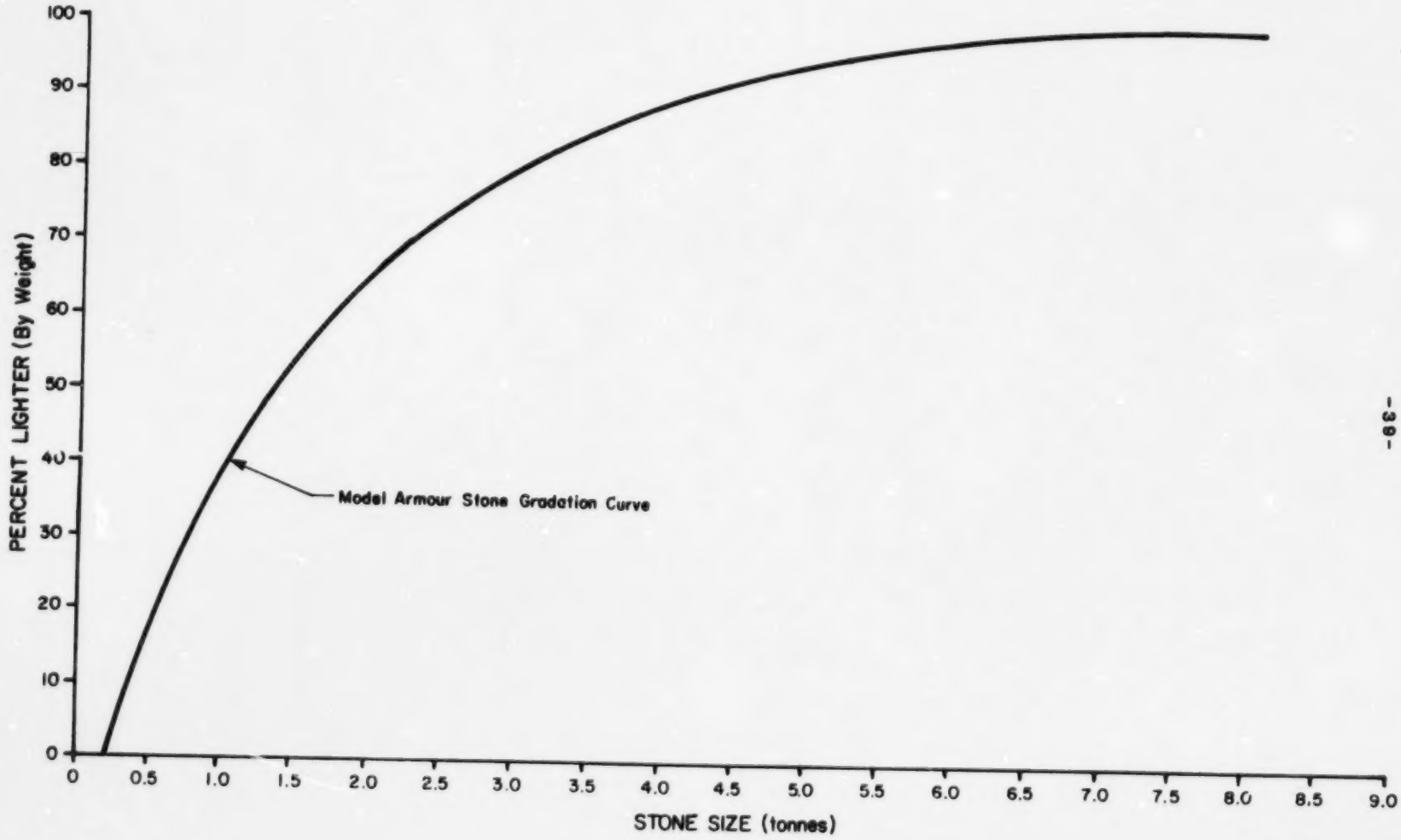


FIGURE 4.6

STEPHENVILLE SOUTH BREAKWATER
MODEL STONE GRADATION CURVE - BERM DESIGN TESTS

end of stage 10. Figures 4.7.2 to 4.7.4 clearly illustrate the nature of the failures in the berm design breakwater model tests. Note that the core template is clearly visible in Figure 4.7.2 and the degree of core exposure for tests 1 and 2 is shown in Figures 4.7.3 and 4.7.4 respectively.

Figure 4.4 also shows the intial and final DWL (+2.5 m) contour line on the breakwater head. As can be seen from these contour lines (and from the profiled sections; Appendix C), the front face of the breakwater head receeded in a uniform manner and a deposition of armour stones occurred on the south side berm forming a non-uniform ridge-like formation shown in Figure 4.7.5 (station -105°). This material was transported from the developed front face of the breakwater head.

It became apparent from the two berm design tests that a 20 m width of berm at the head of the structure is insufficient for correct development of the berm profile, yet a 10 m berm along the trunk appears to be adequate.

Due to time restraints, further berm design model breakwater tests were not conducted.

4.4 Recommended South Breakwater

It is recommended that a conventional structure based on the parameters used in test 4 (Section 4.2.1) be adopted for the proposed south breakwater. Since overtopping is of little consequence on the south breakwater, it is proposed that the crest width can safely be reduced to 5 m for design purposes.

The two berm design breakwater model tests failed to produce a stable structure, thus the design cannot be considered as an alternative without further model testing. (It will be shown in Chapter VI that the berm design breakwater which failed in test 2 has higher cost estimate than that of the proposed structure and, therefore further model testing of berm structures cannot be justified.)

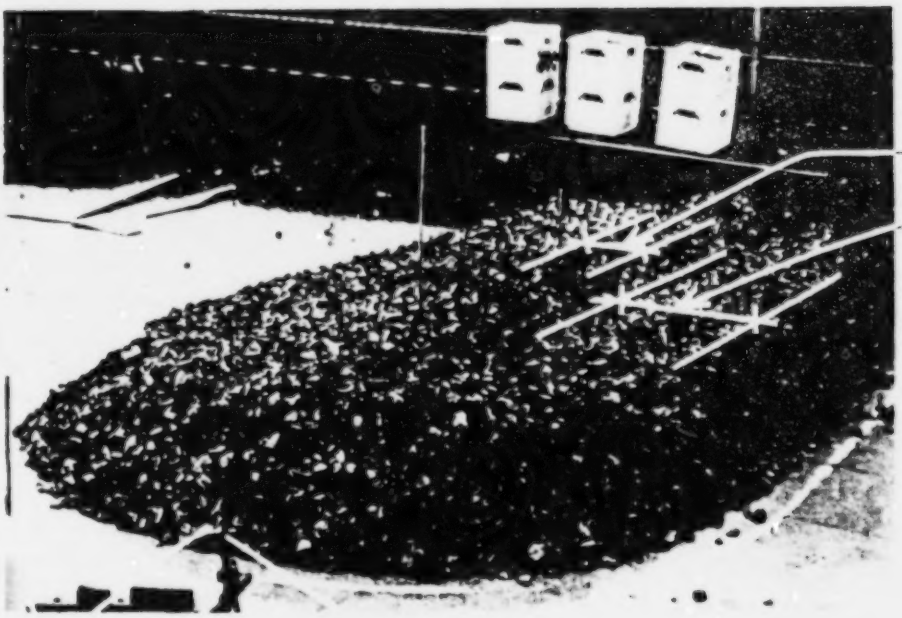


FIGURE 4.7.1

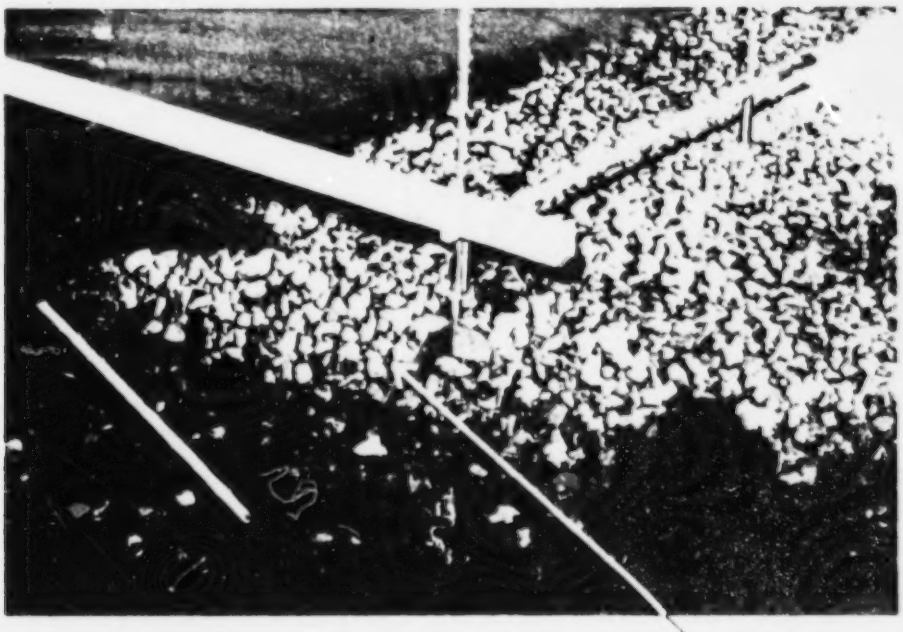


FIGURE 4.7.2

BERM DESIGN TEST 1 - BEFORE AND AFTER PHOTOGRAPHS

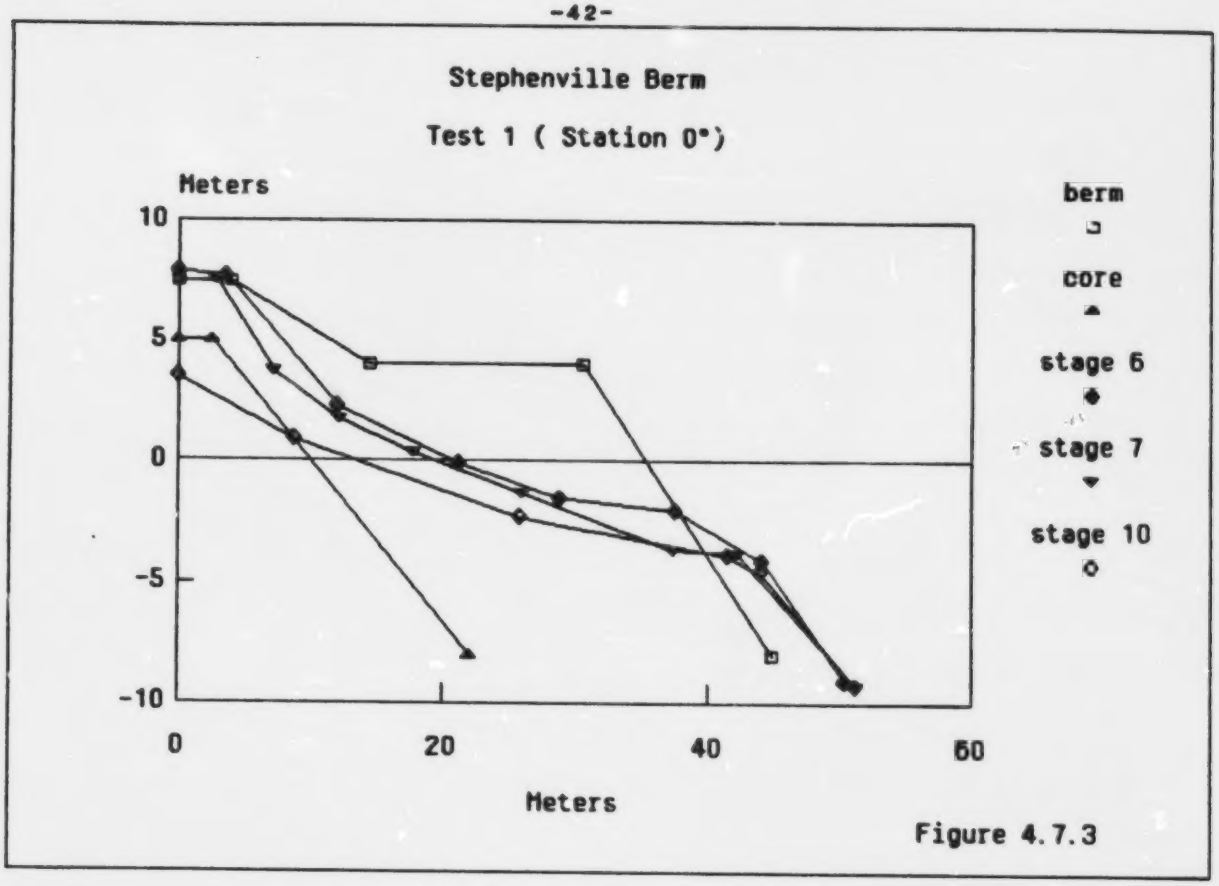


Figure 4.7.3

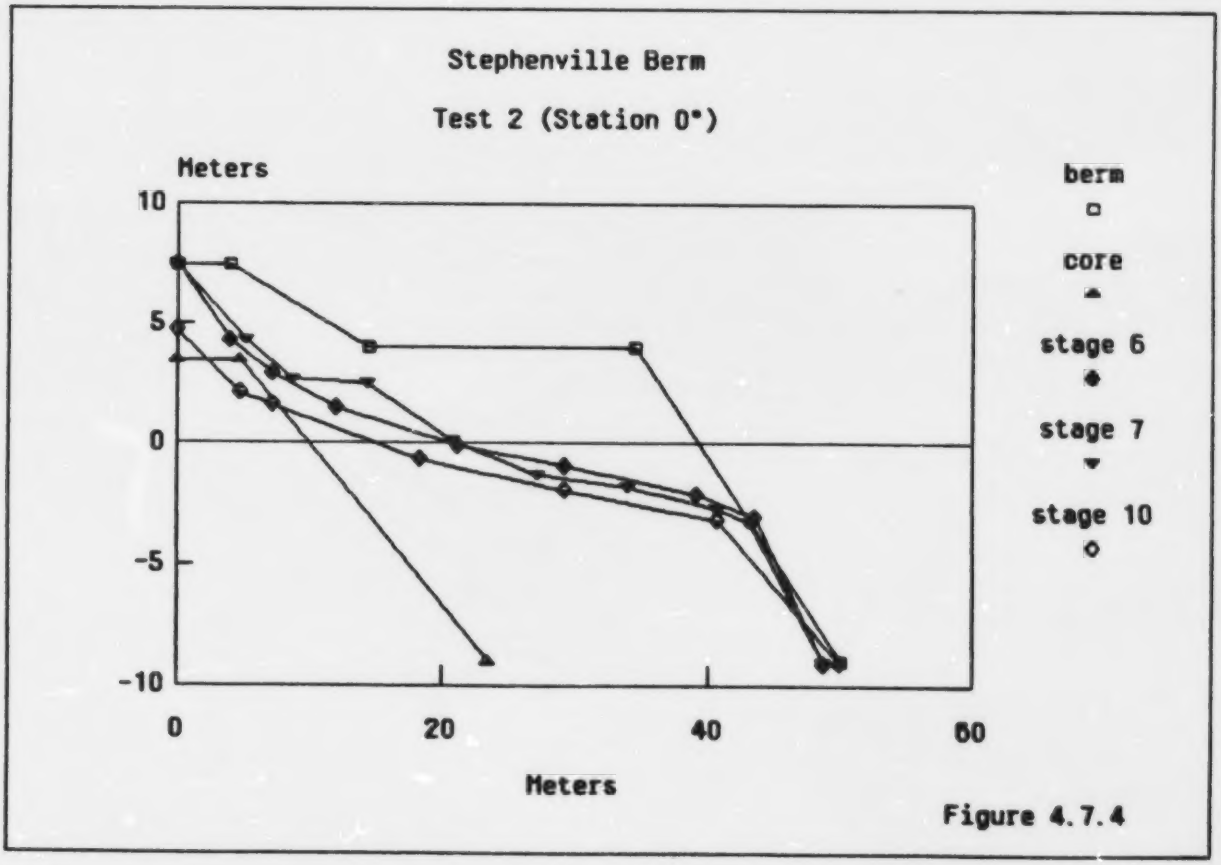


Figure 4.7.4

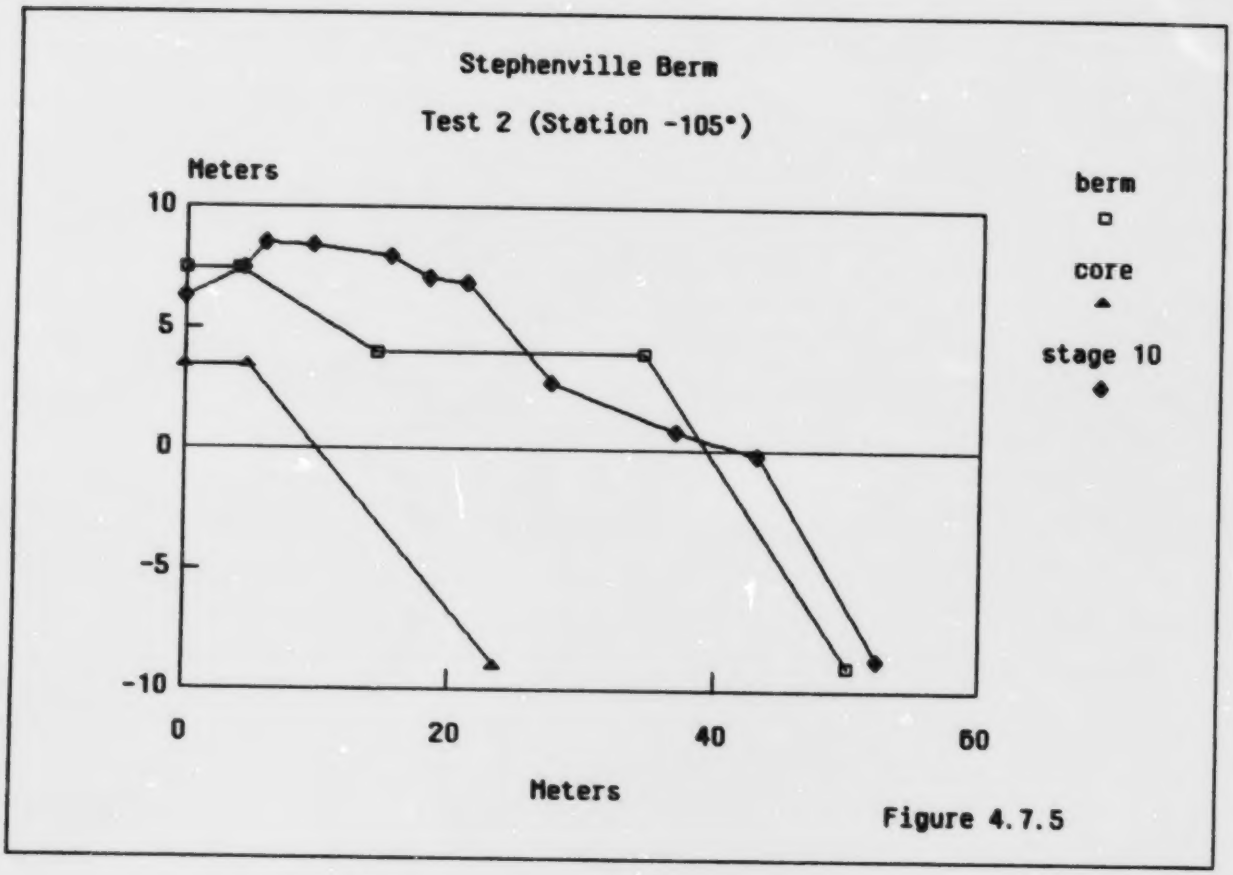

Stephenville Berm
Test 2 (Station -105°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 10

Figure 4.7.5

The recommended design layout and sections based on the above are shown in Figures 4.8 and 4.9 respectively. Note from Figure 4.8 that four lengths of breakwater are presented. These lengths are such that the toe of the breakwater head is located at the 8 m, 7 m, 6 m and 5 m contour line. This variable length of breakwater will be further discussed in Chapter V.

The dimensions that are shown in Figure 4.8 are for the breakwater extending to the 8 m contour line. The parameters shown for the head of the structure in Figure 4.8 apply to the head of any length of structure up to the 8 m contour line. It should be noted that on the north side of the breakwater, the western 30% of trunk length consists of 2^t-10^t armour and the remaining 70% consists of 2^t-5^t armour. These proportions of trunk armour on the north side apply to any length of structure up to the 8 m contour line. The quantities and costs associated with these four schemes of the proposed structure are presented in Chapter VI (proposal 1).

For cost comparison purposes, the quantities and costs associated with two berm design breakwaters are also presented in chapter VI. The first (proposal 2) lists the quantities of material required for the 20 m tested berm section shown in Figure 4.4. The second (proposal 3), lists the quantities for a hypothetical stable ("fat-head") berm design structure (Figures 4.10 and 4.11) having a berm width and a berm elevation at the head of 30 m and 7.5 m respectively.[2] This "fat-head" berm design structure HAS NOT been model tested, but it has proved successful at other locations with similar conditions. For both proposals 2 and 3, the trunk berm width decreases down to 10 m from their respective values at the head.

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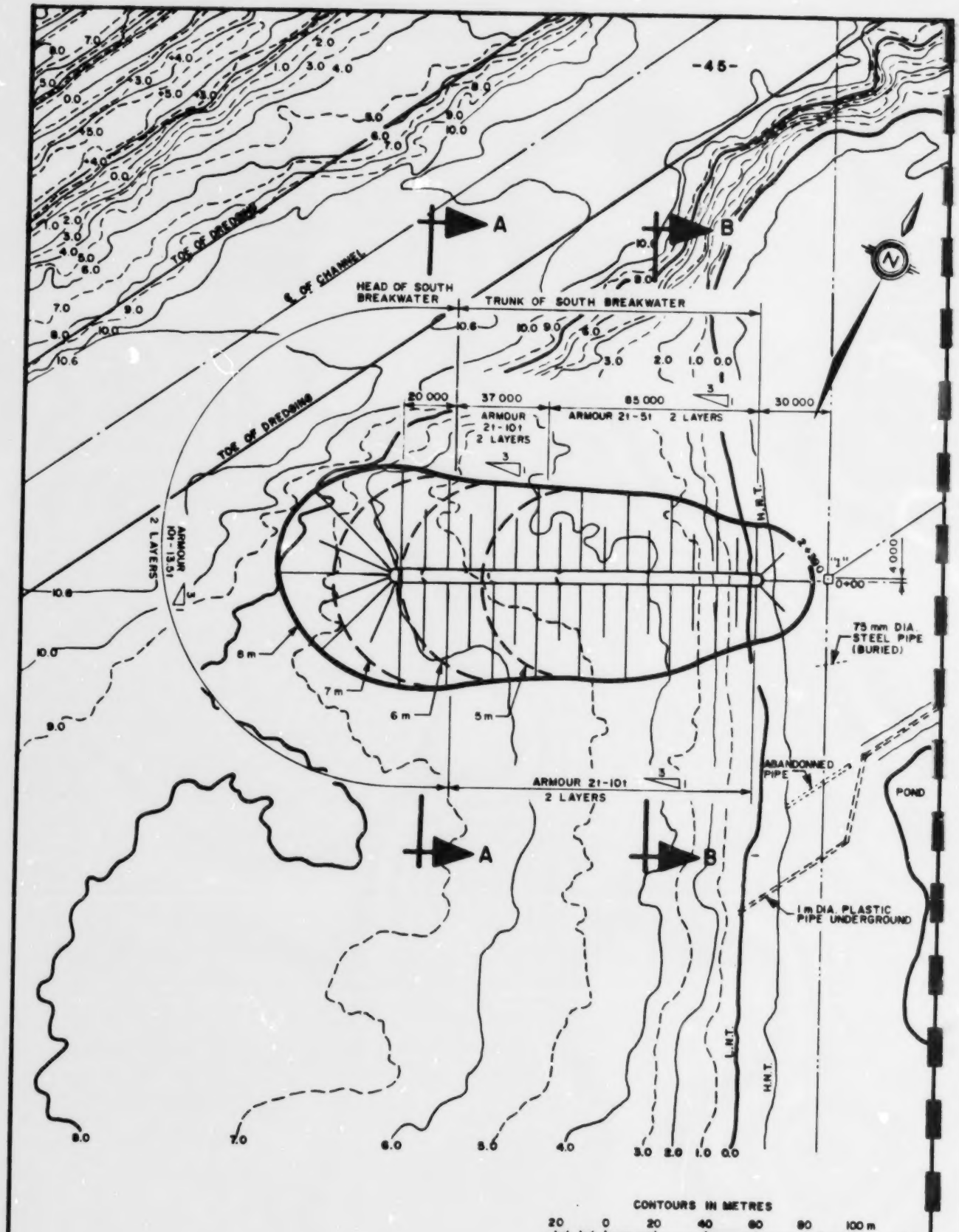
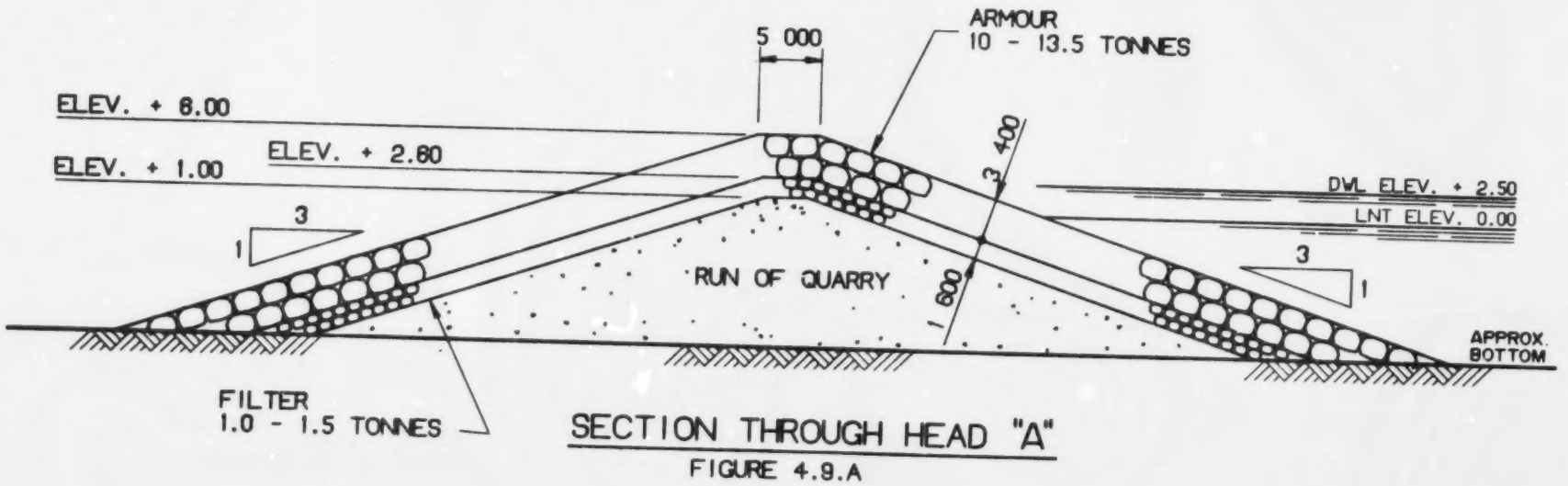
[2] It is believed that this "fat-head" structure would survive the berm design "storm profile" (discussed in Section 4.3) without exposing the core of the structure at any location on the breakwater.



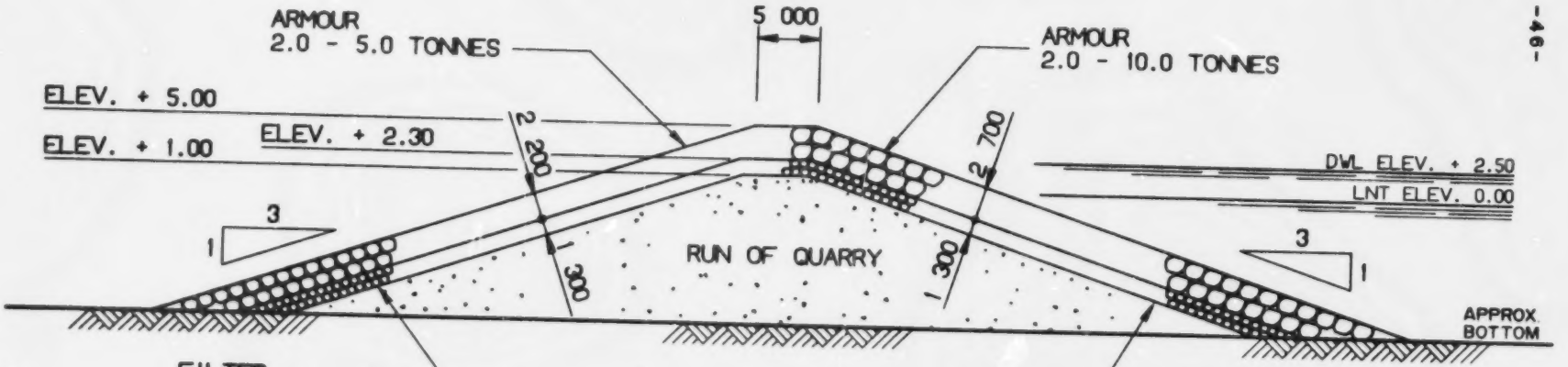
STEPHENVILLE SOUTH BREAKWATER
LAYOUT OF PROPOSED STRUCTURE
(EXTENDING TO 8, 7, 6 AND 5 m DEPTH OF WATER AT TOE OF HEAD)

FIGURE 4.8



SECTION THROUGH HEAD "A"

FIGURE 4.9.A



SECTION THROUGH TRUNK "B"

FIGURE 4.9.B

NOTE :

ALL DIMENSIONS IN MILLIMETERS

SCALE 1:500

STEPHENVILLE, NFLD.

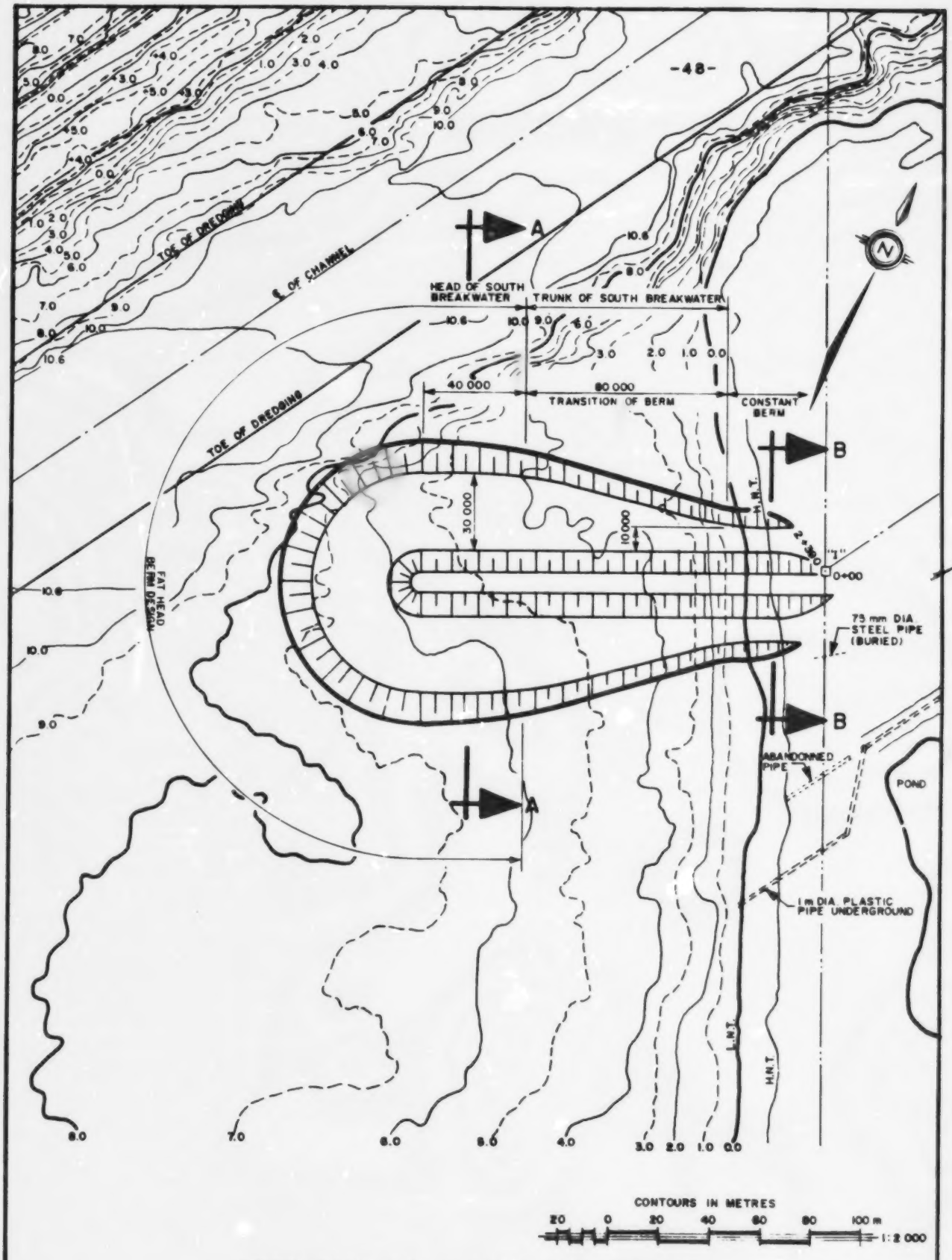
SECTIONS THROUGH HEAD & TRUNK

FIGURES 4.9.A & 4.9.B

4.5 Site Visit and Quarry Inspection

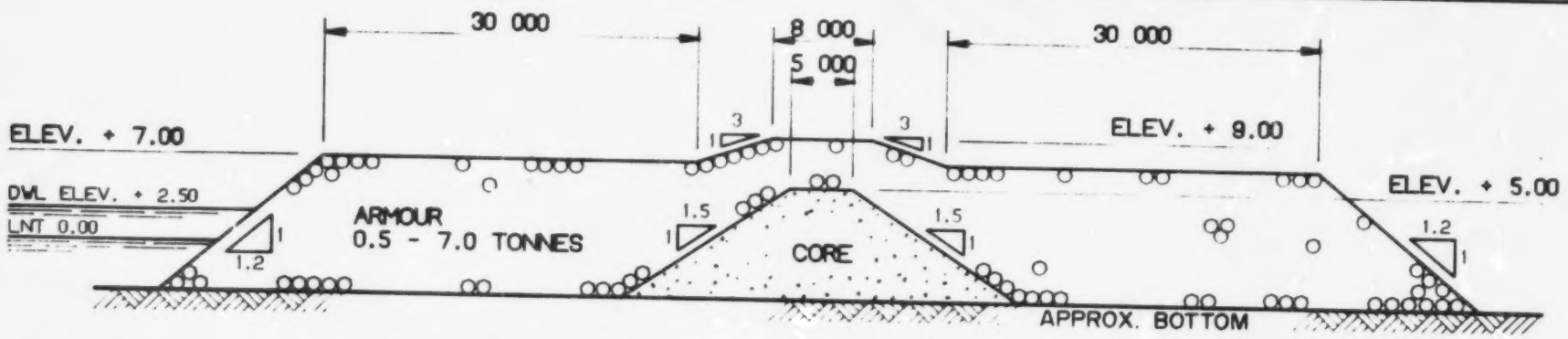
On November 13, 1985, a site visit to Stephenville was made by P.W.C. personnel. During the visit, an inspection of the Indian Head quarry (see Figure 1.2) operation was made which was actively producing armour stone for the north breakwater reconstruction project. From the observations made then and from the discussions held with the quarry operators, it is believe that the Indian Head quarry can readily produce the size of armour stone required for the proposed south breakwater (conventional) structure recommended in Section 4.4.

The North Breakwater reconstruction project was also inspected. It is of interest to note that while standing on the head of the structure, it was observed that the shoaling and breaking patterns of the incoming wave train were similar to those observed in the model study. Furthermore, the rushing mass of water around the breakwater head as described in Section 3.3.3 was also quite visible and is depicted on the cover picture to this report.



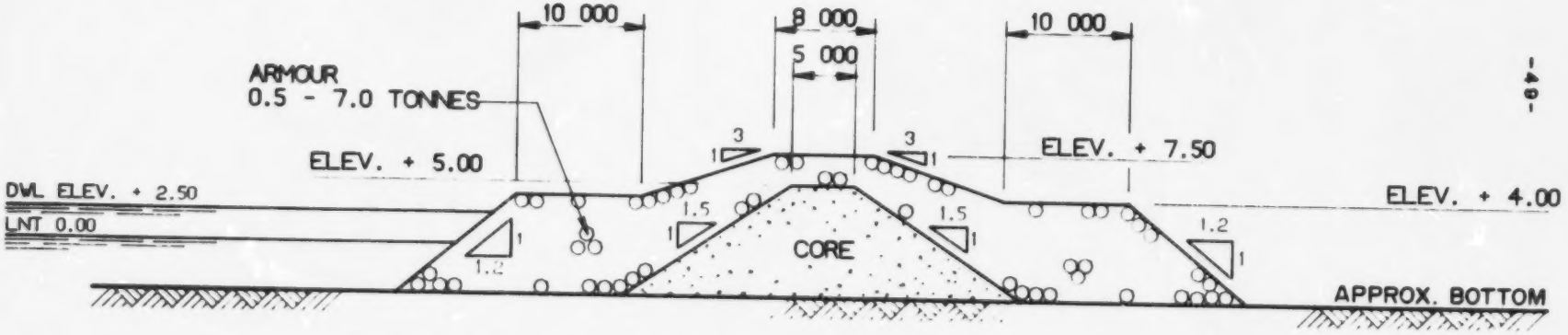
STEPHENVILLE SOUTH BREAKWATER
LAYOUT OF FAT HEAD BERM DESIGN (NOT TESTED)

FIGURE 4.10



SECTION THROUGH FAT HEAD "A" (Berm 30 m)

FIGURE 4.11.A



SECTION THROUGH TRUNK "B" (Berm 10 m)

FIGURE 4.11.B

FIGURES 4.11.A & 4.11.B

NOTE :

ALL DIMENSIONS IN MILLIMETERS

STEPHENVILLE, NFLD.

SECTIONS THROUGH FAT HEAD BERM DESIGN (Not Tested)

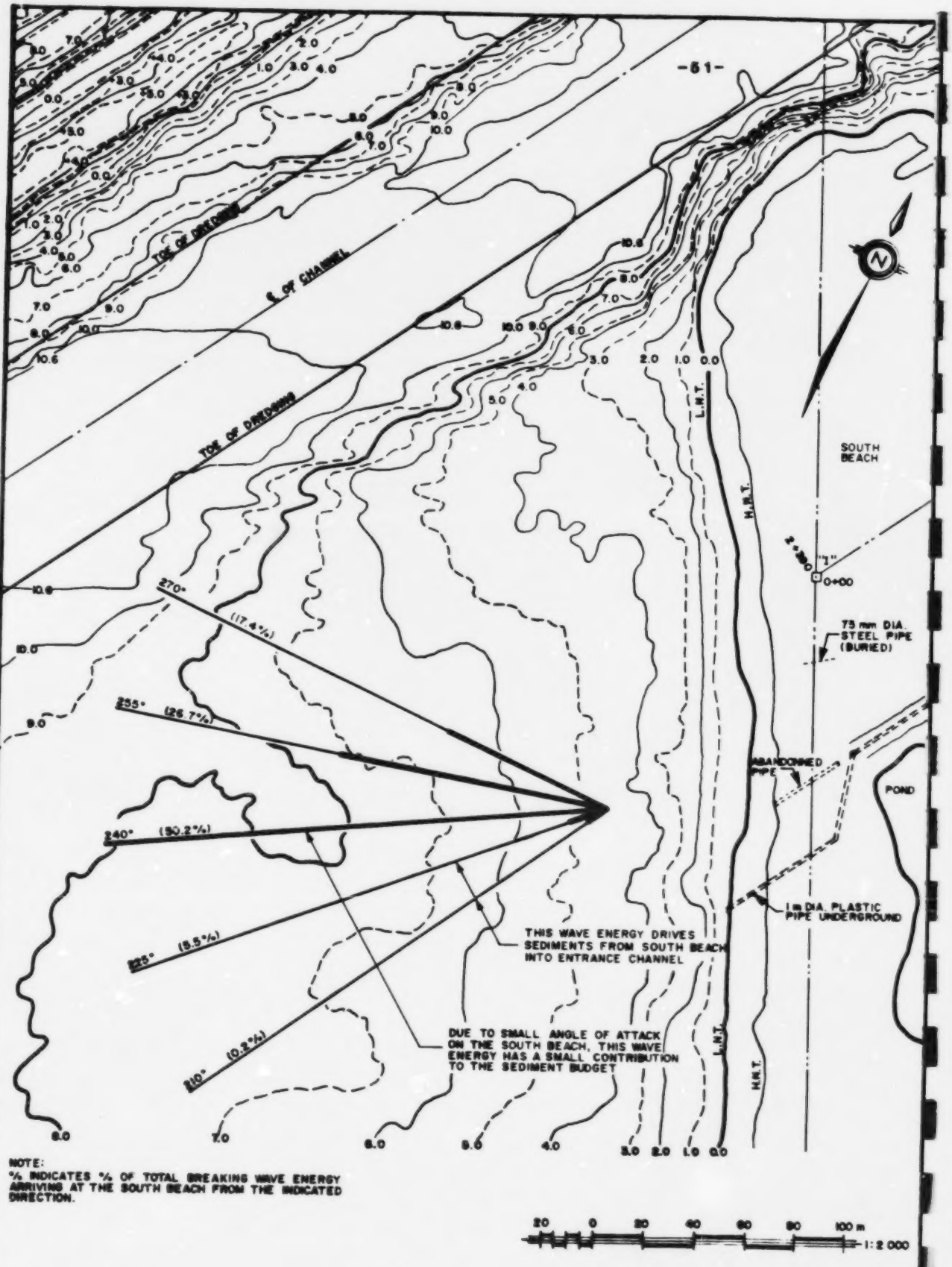
SCALE 1:500

V SEDIMENTATION

It was concluded in [1] that sand and gravel is being transported from the south beach and deposited off the end of the beach forming a spit which migrates into the entrance channel. Sieve analysis were performed on the material located on the south beach and that material which was deposited in the entrance channel (Appendix E). A comparison of the gradation curves indicates a strong resemblance of the two materials (giving evidence to the above claim).

Figure 5.1 shows the breaking wave energy rose located approximately halfway along the south beach based on south beach breaking wave climate presented in [1]. It is apparent from this figure, that a portion of the total breaking wave energy can transport sediments into the entrance channel. That is, approximately 5% to 15% of the total breaking wave energy will transport material towards the entrance channel.

WFBA also conclude from their study that fine sand is being transported south from the north beach around the breakwater head and is subsequently being deposited on south side of the existing breakwater. An examination of gradation curves of material from this location confirms that it is finer than that being deposited off the south beach. This indicates that there are two distinct sources of material (i.e., north beach and south beach) for the two locations of deposition (i.e. off the head of existing breakwater and off the south beach). Furthermore, it was reported in [1] that the net energy vector (which is representative of the net direction of sediment transport in an open system) is in the order of 255°. This energy vector has a southerly component of energy transfer in relation to the shoreline. Therefore, it can be concluded from the above, that wave energy transfer and thus sediment transport is generally to the south. Nevertheless, there is energy available which can drive sediments in a northerly direction, i.e. from the south beach and towards in the entrance channel.



NOTE:
% INDICATES % OF TOTAL BREAKING WAVE ENERGY ARRIVING AT THE SOUTH BEACH FROM THE INDICATED DIRECTION.

BREAKING WAVE ENERGY ROSE ON SOUTH BEACH

FIGURE 5.1

Table 5.1 shows the actual annual siltation rates (m^3/yr) off the south beach, off the head of the north breakwater and the total annual siltation rate in the entrance channel. Both WFBA and PWC used pre and post dredging sounding surveys for the calculations.

TABLE 5.1

Annual rates (m^3/yr) of infilling in entrance channel

Row	Measurement Period	Off south beach in entrance channel 2+230 to 2+410	Off head of north breakwater in entrance channel 2+560 to 2+680	Total infilling in entrance channel 2+100 to 2+680
1	WFBA [1] (1953-1958)	5,550	3,000	24,900
2	WFBA [1] (1958-1972)	3,500	913	8,550
3	WFBA [1] (1975-1980)	8,070	2,300	19,000
4	PWC (1981-1985)	6,000	N/A	N/A

The quantities listed in row 2 are significantly lower than those in other rows because dredging records indicate that the entrance channel had infilled and required dredging by 1963 which was not undertaken until 1971 and 1972. Therefore, from 1963 to 1971, it is believed that a state of equilibrium with respect to the sediment regimes may have prevailed in the entrance channel. Therefore, the estimate (row 2) is an average siltation rate over a long period of time with no dredging events. In contrast, rows 1 and 3 represent ("dynamic") infilling rates measured over a shorter period of time and just after a dredging event.

As previously mentioned (Section 4.1), WFBA claim the south breakwater should extend to the -8 m contour line in order to prevent infilling in the entrance channel near the south beach. Here it is suggested that the length of breakwater required could be reduced if some infilling off the south beach could be tolerated. Using the breaking wave climate on the south beach [1] and sediment characteristics located there (Appendix E), the initiation of sediment motion was analysed to estimate the reduction of infilling rate off the south beach as a function of breakwater length. The values obtained are listed in Table 5.2.

TABLE 5.2

Percentage Reduction in Sedimentation off South Beach as a Function of Breakwater Length

Depth of water at toe of breakwater	5 m	6 m	7 m	8 m
Approx. length of breakwater	78 m	104 m	132 m	142 m
Estimated reduction in sedimentation off south beach (%R)	50%	70%	80%	90%

The calculations were based on the techniques outlined in [12], [13], and [15]. The estimates were arrived at by first calculating the critical wave height Hcr (and wave period) required to initiate sediment transport of the existing material in a given depth of water. The percentage of exceedance of Hcr (%H > Hcr)[1] was then determined from the wave climate presented in [1]. Finally the percentage reduction (%R) of infilling was assumed to be

$$(\%R) = 100 - (\%H > Hcr)$$

For example, in a depth of 8 m, the critical wave height Hcr that causes the sediments to initiate motion at that depth is exceeded 10% of the time. Thus at this depth, a breakwater structure would reduce sedimentation by approximately 90%. It should be clearly realized that the above technique is only an approximation. A more in-depth analysis of the sedimentation process would require numerical and/or physical modelling of these processes.

---

[1] Only those directions which transport sediments towards the entrance channel were used in the calculation of H > Hcr.

To estimate the effects of construction of the south breakwater on the dredging cycle, it is suggested that the reduced rate of infilling off the south beach be estimated using Tables 5.1 and 5.2. This information can subsequently be used to estimate the dredging requirement corresponding to each of the lengths of breakwater proposed and to conduct a cost evaluation of the various schemes.

VI QUANTITIES AND COSTS: SOUTH BREAKWATER

In this chapter, quantities and cost estimates of the proposed structures in Section 4.4 are presented. Unit prices were based on latest bid unit prices and discussions held with local contractors.

i) Proposal 1 - Table 6.1 lists the quantities and cost estimate for the recommended structure (of variable length) discussed in Section 4.4 and shown in Figures 4.8 and 4.9;

ii) Proposal 2 - Table 6.2 lists the quantities and cost estimate required for the berm design structure discussed in Section 4.3 and shown in Figures 4.4 and 4.5 which failed in test 2.

iii) Proposal 3 - Table 6.3 lists the quantities and cost estimate for the hypothetical stable (not yet tested) "fat-head" berm design structure discussed in Section 4.4 and shown in Figures 4.10 and 4.11.

TABLE 6.1
Quantities and Cost Estimates for a Conventional Structure of Variable Length
See Figures 4.8 and 4.9
Proposal 1

Material	Range tonnes	Unit Price $/t	Depth at Toe of Head 5 m (tonnes)	6 m (tonnes)	7 m (tonnes)	8 m (tonnes)
Armour	10^t-13.5^t	30	15,700	18,950	21,800	23,300
Armour	2^t-10^t	21	5,800	11,000	15,500	17,700
Armour	2^t- 5^t	20	3,900	7,250	10,100	11,600
Filter	1.0^t-1.5^t	15	4,700	5,700	6,800	7,350
Filter	0.2^t-1.0^t	12	3,200	6,650	9,500	10,950
Core		8	11,600	22,700	32,000	36,050
Total			$ 872,500	$1,291,400	$1,653,500	$1,832,750

TABLE 6.2
Quantities and Cost Estimate for
Berm Design Structure (Test 2)
See Figures 4.4 and 4.5
Proposal 2

Material	Stone Range	Tonnes	Price $/t	$
Armour	0.5t-7.0t	165,368t	15	2,480,520
Core		54,472t	8	435,770
TOTAL		219,840t		2,916,300

Depth at toe = 8 m

TABLE 6.3
Quantities and Cost Estimate for
Berm Design Structure (Fat-Head Section)
See Figures 4.10 and 4.11
Proposal 3

Material	Stone Range	Tonnes	Price $/t	$
Armour	0.5t-7.0t	260,286	15	3,904,290
Core		53,680	8	429,410
TOTAL		313,966		4,333,700

Depth at toe = 8 m

Note - costs of mobilization are not included.

VII CONCLUSIONS

i) For stability, the head of the north breakwater reconstruction requires 7^t-10^t armour stone placed at a 3:1 slope. The trunk is stable when utilizing 2^t-10^t armour stone placed at a 3:1 slope. A stone apron is required on the south side of the head to stabilize the side slope, mitigate scour and thus prevent sliding of armour stones into the entrance channel. Dredging of the approach (remnants of former structure) to -5 m would alter the breaking wave pattern such that the breaking wave energy is more spread out and the structure is more stable. However, as mentioned in Section 3.3.3, dredging to -5.0 m is not considered necessary.

ii) For stability, the head of the south breakwater requires 10^t-13.5^t armour stone placed at a 3:1 slope. The trunk is stable when utilizing 2^t-10^t armour stone placed at a 3:1 slope. A breakwater extending out from the south beach will reduce the siltation rate off the end of the south beach proportionally to the length of that breakwater adopted. The construction of the south breakwater will not reduce the rate of sediment transport of sand from the north beach which is infilling the entrance channel.

iii) Dredging to -10.6 m and straightening (re-alignment) of the entrance channel is required for safe navigation.

iv) It is recommended to conduct a cost evaluation for the various lengths of south breakwater (Section 4.4) and their cost effectiveness in order to determine the optimum length of breakwater.

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APPENDIX A

Summary of wave characteristics recorded in North Breakwater model study

North Breakwater Model Study

Summary of Wave Characteristics Recorded During Test 1

Storm Profile Stage	Wave Gauge (See Figure 3.2)	Significant Wave Height $H_{1/3}$ (m)	Maximum Wave Height H_{max} (m)	Spectral Peak Period T_p (s)
1	1	3.0	4.8	8.0
	2	2.4	3.6	8.0
	3	2.6	3.6	8.0
	4	2.1	2.8	8.0
2	1	4.0	6.1	9.0
	2	3.5	5.1	9.2
	3	3.8	5.8	9.3
	4	2.8	4.0	9.3
3	1	4.9	7.2	9.9
	2	4.4	6.2	10.1
	3	5.1	7.2	10.2
	4	3.2	4.5	10.2
4	1	5.9	9.7	10.8
	2	4.7	6.6	10.9
	3	5.9	7.4	11.0
	4	3.4	4.4	11.2
5	1	6.1	9.5	11.7
	2	5.1	6.7	11.8
	3	6.6	8.3	12.1
	4	3.5	4.1	12.0

North Breakwater Model Study

Summary of Wave Characteristics Recorded During Test 2

Storm Profile Stage	Wave Gauge (See Figure 3.2)	Significant Wave Height $H_{1/3}$ (m)	Maximum Wave Height H_{max} (m)	Spectral Peak Period T_p (s)
1	1	3.0	5.0	8.0
	2	2.4	3.7	8.1
	3	2.7	3.9	8.0
	4	2.2	3.1	8.0
2	1	4.0	6.5	8.9
	2	3.5	5.4	9.1
	3	4.0	6.4	9.1
	4	2.8	4.1	9.2
3	1	5.1	7.6	10.0
	2	4.5	6.2	10.1
	3	5.1	7.0	10.2
	4	3.2	4.5	10.2
4	1	5.9	9.3	10.8
	2	4.7	6.7	10.9
	3	6.0	8.0	11.0
	4	3.4	4.4	11.3
5	1	6.2	9.8	11.7
	2	4.6	6.6	11.9
	3	6.7	8.1	12.1
	4	3.8	6.9	11.9

North Breakwater Model Study

Summary of Wave Characteristics Recorded During Test 3

Storm Profile Stage	Wave Gauge (See Figure 3.2)	Significant Wave Height $H_{1/3}$ (m)	Maximum Wave Height H_{max} (m)	Spectral Peak Period T_p (s)
1	1			
	2			
	3			
	4			
2	1	3.5	5.4	8.9
	2	2.8	4.5	9.3
	3	3.2	4.6	9.2
	4	1.8	3.1	9.2
3	1	5.1	7.2	9.9
	2	3.8	5.3	10.0
	3	4.0	5.8	10.2
	4	1.8	2.6	10.4
4	1	5.6	8.9	11.0
	2	4.0	5.2	11.1
	3	4.2	5.8	11.1
	4	1.9	3.0	11.8
5	1			
	2			
	3			
	4			

North Breakwater Model Study

Summary of Wave Characteristics Recorded During Test 4

Storm Profile Stage	Wave Gauge (See Figure 3.2)	Significant Wave Height $H_{1/3}$ (m)	Maximum Wave Height H_{max} (m)	Spectral Peak Period T_p (s)
1	1	2.9	4.9	8.0
	2	2.3	3.8	8.1
	3	2.6	3.8	8.0
	4	1.9	2.6	8.0
2	1	4.2	6.5	8.9
	2	3.5	5.2	9.2
	3	3.9	5.6	9.1
	4	2.8	4.2	9.2
3	1	5.0	7.5	10.0
	2	4.5	6.4	10.0
	3	4.9	6.5	10.2
	4	3.2	5.0	10.3
4	1	5.9	9.8	10.8
	2	4.8	6.3	10.9
	3	5.7	7.3	11.0
	4	3.9	5.2	11.3
5	1	6.0	9.2	11.7
	2	5.1	6.9	11.7
	3	6.4	7.8	12.1
	4	4.0	5.4	11.8

North Breakwater Model Study

Summary of Wave Characteristics Recorded During Test 5

Storm Profile Stage	Wave Gauge (See Figure 3.2)	Significant Wave Height $H_{1/3}$ (m)	Maximum Wave Height H_{max} (m)	Spectral Peak Period T_p (s)
1	1	2.9	4.6	8.0
	2	2.3	3.8	8.0
	3	2.5	3.8	8.0
	4	1.9	2.8	8.0
2	1	3.9	6.1	8.9
	2	3.4	5.1	9.2
	3	3.6	5.6	9.2
	4	2.7	4.1	9.1
3	1	4.8	7.3	10.0
	2	4.5	6.4	10.2
	3	4.9	6.6	10.2
	4	3.3	4.7	10.3
4	1	5.7	9.0	10.8
	2	4.9	7.0	10.9
	3	5.8	7.3	11.0
	4	3.9	5.0	11.2
5	1	6.2	9.7	11.8
	2	5.3	7.3	11.8
	3	6.4	7.7	12.1
	4	4.1	5.3	11.8

North Breakwater Model Study

Summary of Wave Characteristics Recorded During Test 6

Storm Profile Stage	Wave Gauge (See Figure 3.2)	Significant Wave Height $H_{1/3}$ (m)	Maximum Wave Height H_{max} (m)	Spectral Peak Period T_p (s)
1	1	2.9	4.7	8.0
	2	2.3	3.6	8.1
	3	2.5	3.9	8.1
	4	1.7	2.5	8.0
2	1	3.9	6.0	8.9
	2	3.4	5.5	9.2
	3	3.7	5.7	9.2
	4	3.7	4.0	9.1
3	1	4.9	7.2	10.0
	2	4.4	6.3	10.1
	3	5.1	7.0	10.2
	4	3.4	4.6	10.3
4	1	5.8	9.4	10.8
	2	4.8	6.4	10.8
	3	5.8	7.4	11.0
	4	3.8	4.8	11.2
5	1	N/A	N/A	N/A
	2			
	3			
	4			

North Breakwater Model Study

Summary of Wave Characteristics Recorded During Test 7

Storm Profile Stage	Wave Gauge (See Figure 3.2)	Significant Wave Height $H_{1/3}$ (m)	Maximum Wave Height H_{max} (m)	Spectral Peak Period T_p (s)
1	1 2 3 4	N/A	N/A	N/A
2	1 2 3 4	4.0 3.3 3.7 2.9	6.2 5.3 5.3 4.1	8.9 9.1 9.1 9.2
3	1 2 3 4	5.0 4.4 4.8 3.4	7.2 6.3 6.4 4.9	10.0 10.1 10.2 10.3
4	1 2 3 4	5.7 4.7 5.6 3.8	9.0 6.9 7.2 5.2	10.8 10.9 11.1 11.2
5	1 2 3 4	6.4 5.3 6.7 3.9	9.3 7.3 8.5 5.1	11.8 11.9 12.2 11.8

APPENDIX B

Summary of wave characteristics recorded in South Breakwater model study

South Breakwater Model Study

Summary of Wave Characteristics Recorded During Test 1

Storm Profile Stage	Wave Gauge (See Figure 3.2)	Significant Wave Height $H_{1/3}$ (m)	Maximum Wave Height H_{max} (m)	Spectral Peak Period T_p (s)
1	1	3.0	4.9	8.0
	2	2.7	4.0	8.0
	3	3.0	4.4	8.0
2	1	4.0	6.0	8.8
	2	3.9	6.5	9.2
	3	4.0	6.4	9.0
3	1	4.8	7.8	10.0
	2	5.0	7.3	10.1
	3	4.9	7.0	10.2
4	1	5.7	9.1	10.9
	2	5.7	8.1	10.9
	3	5.6	7.3	10.9
5	1	6.1	9.4	12.3
	2	6.4	8.6	12.1
	3	6.5	8.2	11.9

South Breakwater Model Study

Summary of Wave Characteristics Recorded During Test 2

Storm Profile Stage	Wave Gauge (See Figure 3.2)	Significant Wave Height $H_{1/3}$ (m)	Maximum Wave Height H_{max} (m)	Spectral Peak Period T_p (s)
1	1	2.9	4.8	8.0
	2	2.6	3.9	8.0
	3	2.8	4.0	8.1
2	1	4.0	5.9	8.9
	2	4.0	6.8	9.3
	3	3.8	6.2	9.0
3	1	5.2	7.8	10.0
	2	5.2	8.1	10.1
	3	5.2	7.4	10.2
4	1	6.7	9.5	10.7
	2	5.8	8.5	10.8
	3	5.7	8.0	10.9
5	1	6.2	9.6	11.9
	2	6.5	9.3	12.2
	3	6.1	8.0	11.8

South Breakwater Model Study

Summary of Wave Characteristics Recorded During Test 3

Storm Profile Stage	Wave Gauge (See Figure 3.2)	Significant Wave Height $H_{1/3}$ (m)	Maximum Wave Height H_{max} (m)	Spectral Peak Period T_p (s)
1	1 2 3			
2	1 2 3			
3	1 2 3			
4	1 2 3			
5	1 2 3	6.3 5.5 6.2	10.2 7.5 8.1	12.1 11.9 11.6

South Breakwater Model Study

Summary of Wave Characteristics Recorded During Test 4

Storm Profile Stage	Wave Gauge (See Figure 3.2)	Significant Wave Height $H_{1/3}$ (m)	Maximum Wave Height H_{max} (m)	Spectral Peak Period T_p (s)
1	1	3.4	5.5	8.1
	2	3.1	4.8	7.9
	3	3.4	4.8	8.0
2	1	4.6	7.0	8.8
	2	3.8	6.4	9.1
	3	4.5	7.4	9.0
3	1	5.5	8.7	9.9
	2	4.8	7.6	10.1
	3	5.2	7.7	10.1
4	1	6.2	10.8	10.8
	2	5.4	8.7	11.0
	3	5.8	8.0	11.0
5	1	6.9	10.9	11.9
	2	5.5	7.3	12.0
	3	6.0	7.8	11.0

South Breakwater Model Study (Berm Design)

Summary of Wave Characteristics Recorded During Test 1

Storm Profile Stage	Wave Gauge (See Figure 3.2)	Significant Wave Height $H_{1/3}$ (m)	Maximum Wave Height H_{max} (m)	Spectral Peak Period T_p (s)
1	1	3.0	5.0	8.1
	2	2.5	3.7	8.1
	3	3.1	4.5	8.1
2	1	4.1	6.7	8.8
	2	3.5	6.0	8.9
	3	4.3	6.7	9.0
3	1	5.1	7.9	9.9
	2	4.5	7.4	10.0
	3	5.2	7.2	10.0
4	1	5.9	9.4	10.8
	2	5.4	8.1	11.1
	3	5.9	7.8	11.0
5	1	6.3	9.7	12.0
	2	5.7	7.7	12.0
	3	6.3	7.8	11.8

South Breakwater Model Study (Berm Design)

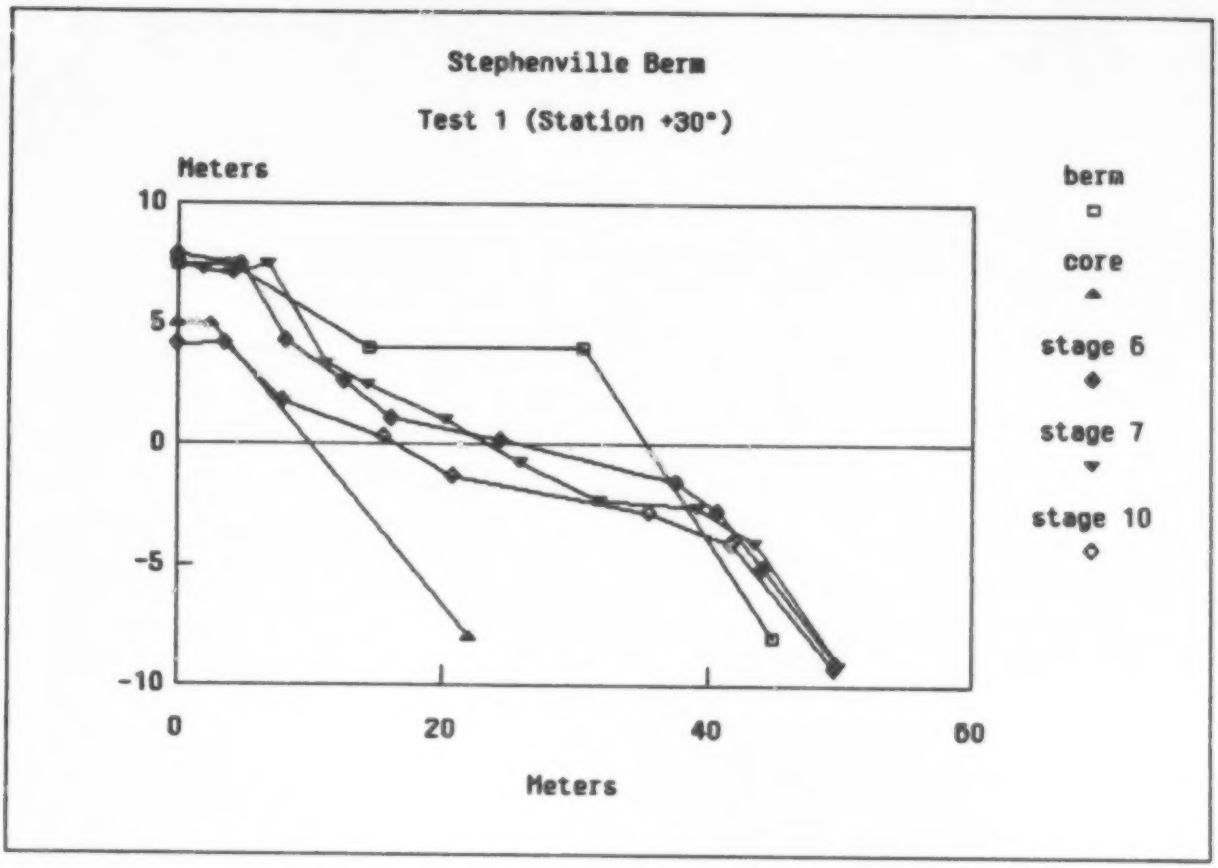
Summary of Wave Characteristics Recorded During Test 2

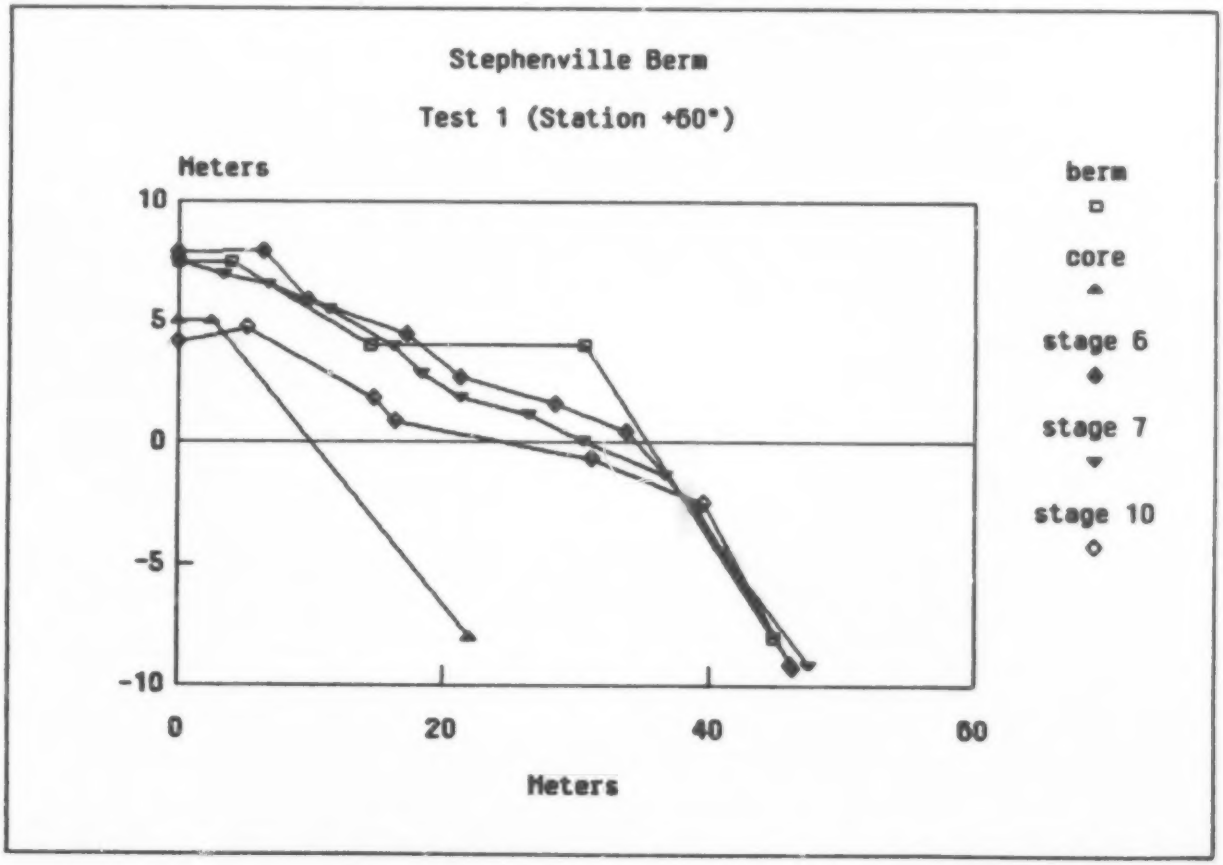
Storm Profile Stage	Wave Gauge (See Figure 3.2)	Significant Wave Height $H_{1/3}$ (m)	Maximum Wave Height H_{max} (m)	Spectral Peak Period T_p (s)
1	1	3.1	5.2	8.1
	2	2.6	3.8	8.0
	3	3.2	4.5	8.1
2	1	4.2	6.4	8.9
	2	3.7	6.4	9.0
	3	4.2	6.5	9.0
3	1	5.1	7.6	9.9
	2	4.6	7.7	10.0
	3	4.8	7.3	10.1
4	1	6.0	9.7	10.7
	2	5.5	8.0	11.1
	3	5.7	7.6	11.1
5	1	6.4	9.8	12.1
	2	6.1	8.3	12.0
	3	6.3	8.5	11.8

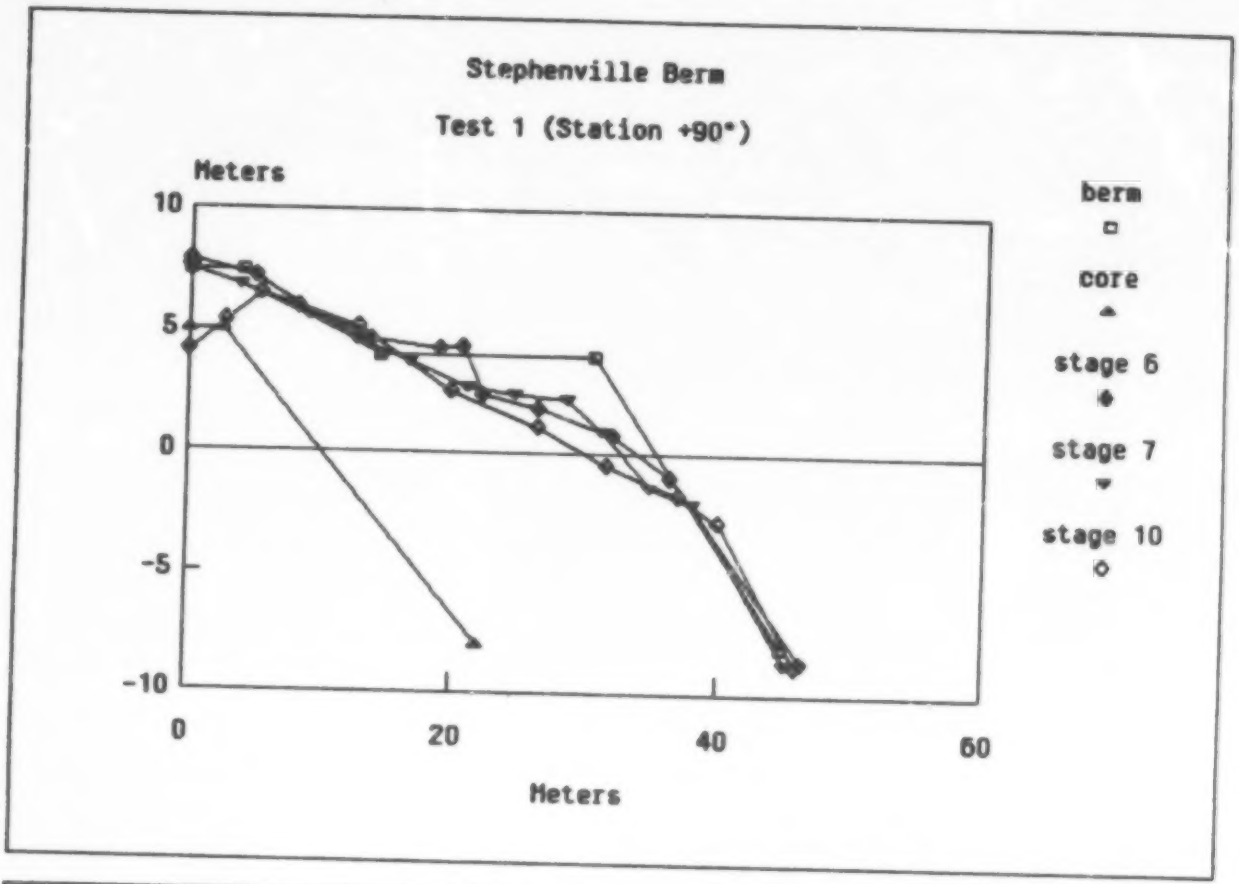
APPENDIX C

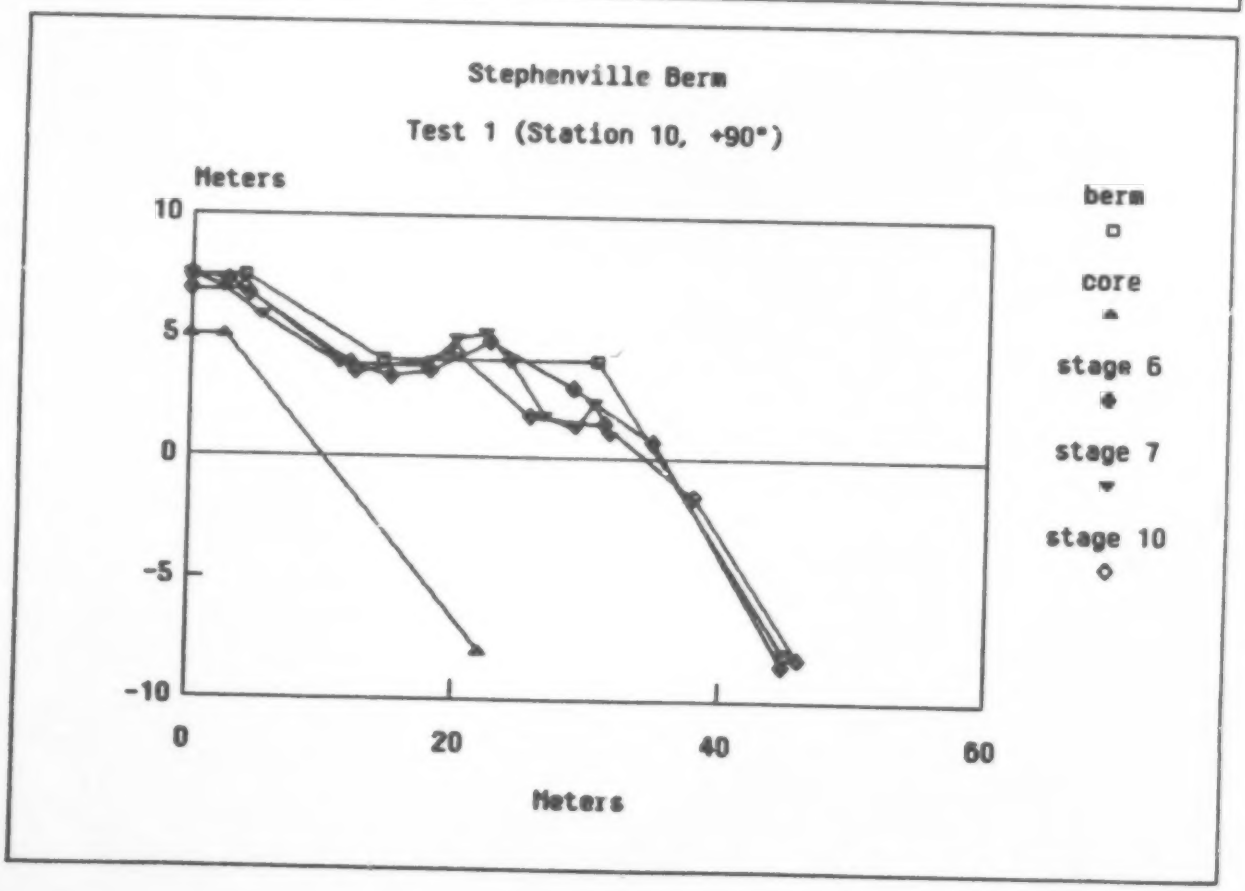
Berm Profiles

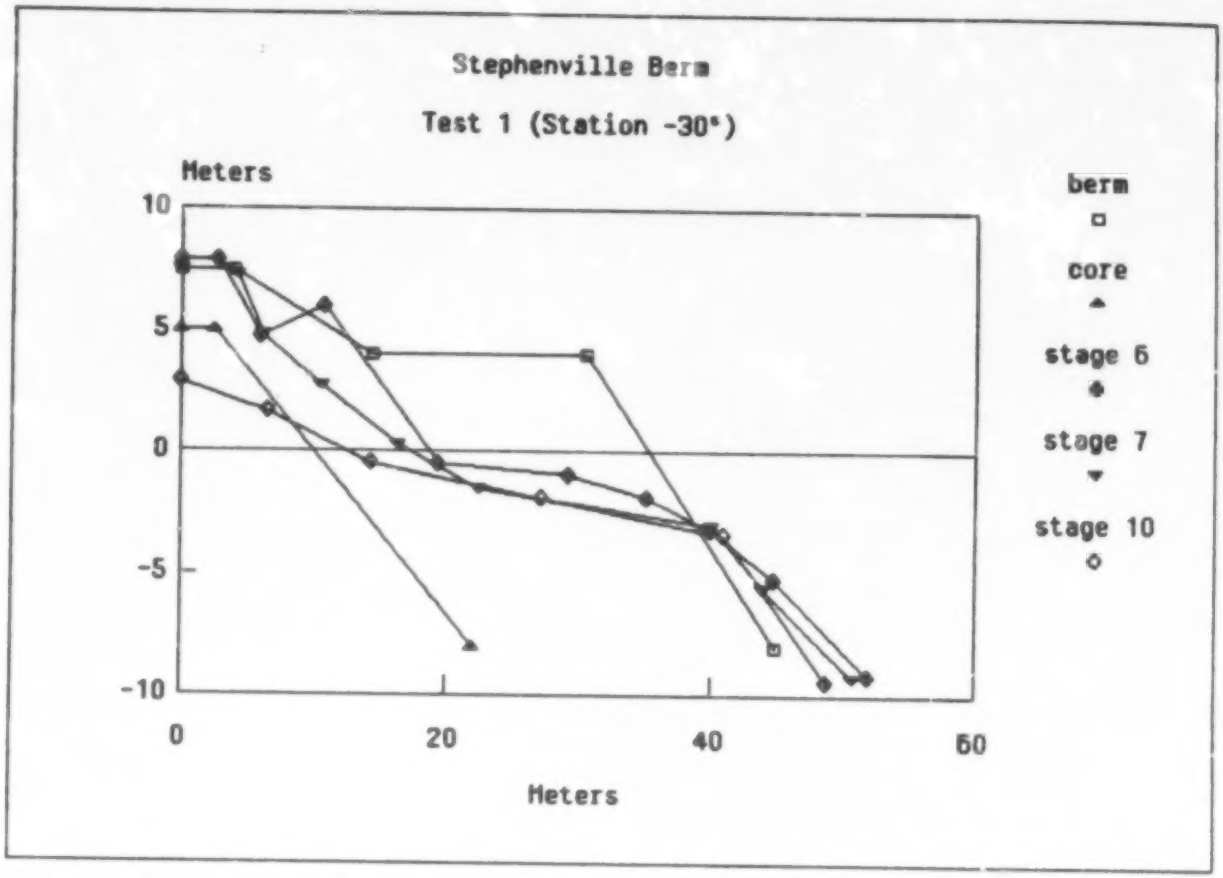
(Stations shown in Figure 4.4)

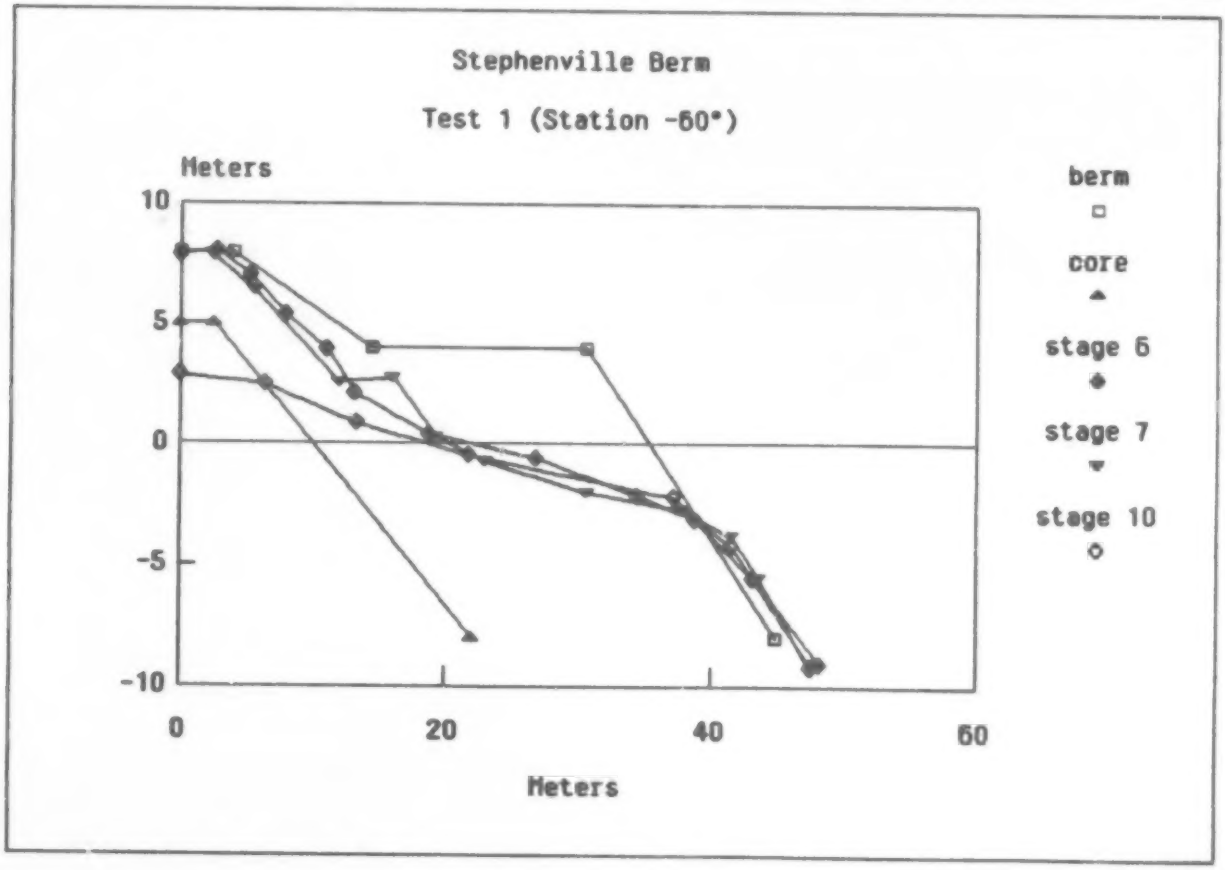

Stephenville Berm
Test 1 (Station +30°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10

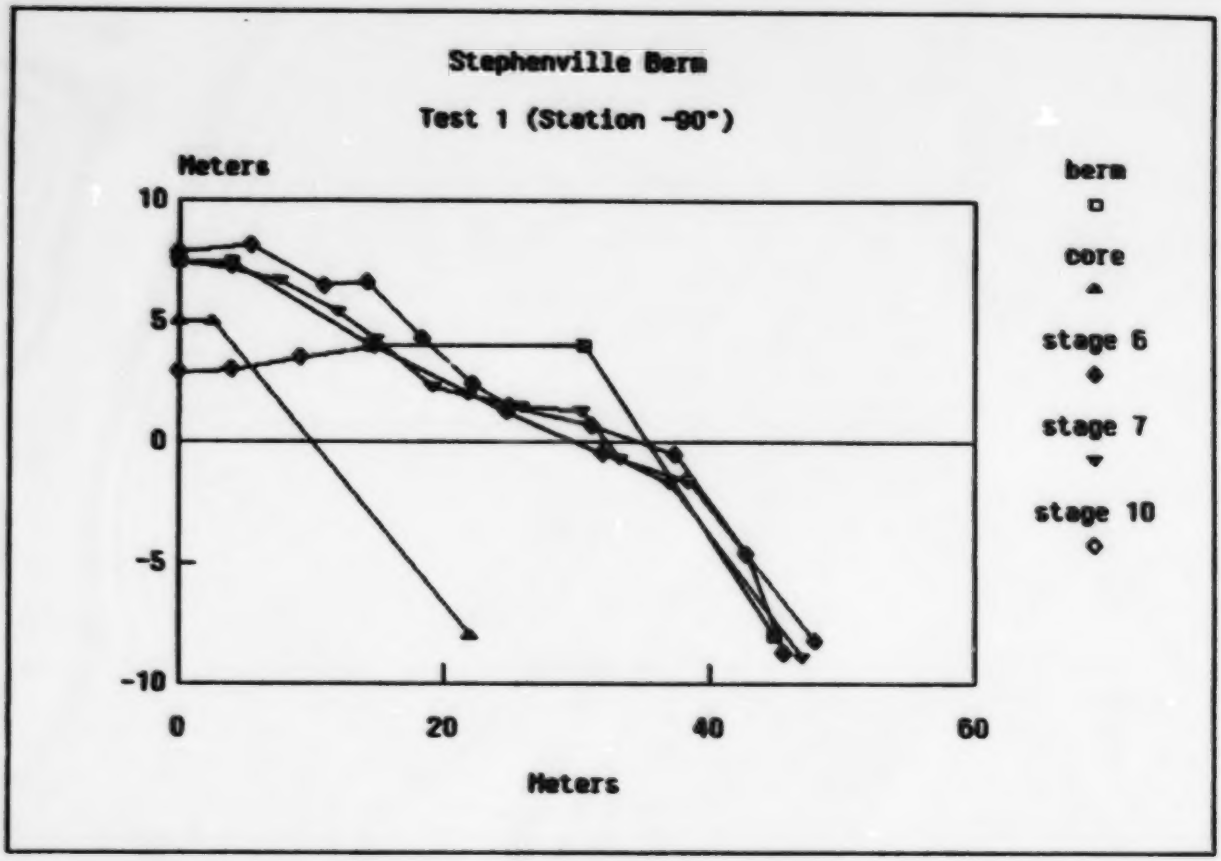

Stephenville Berm
Test 1 (Station +60°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10

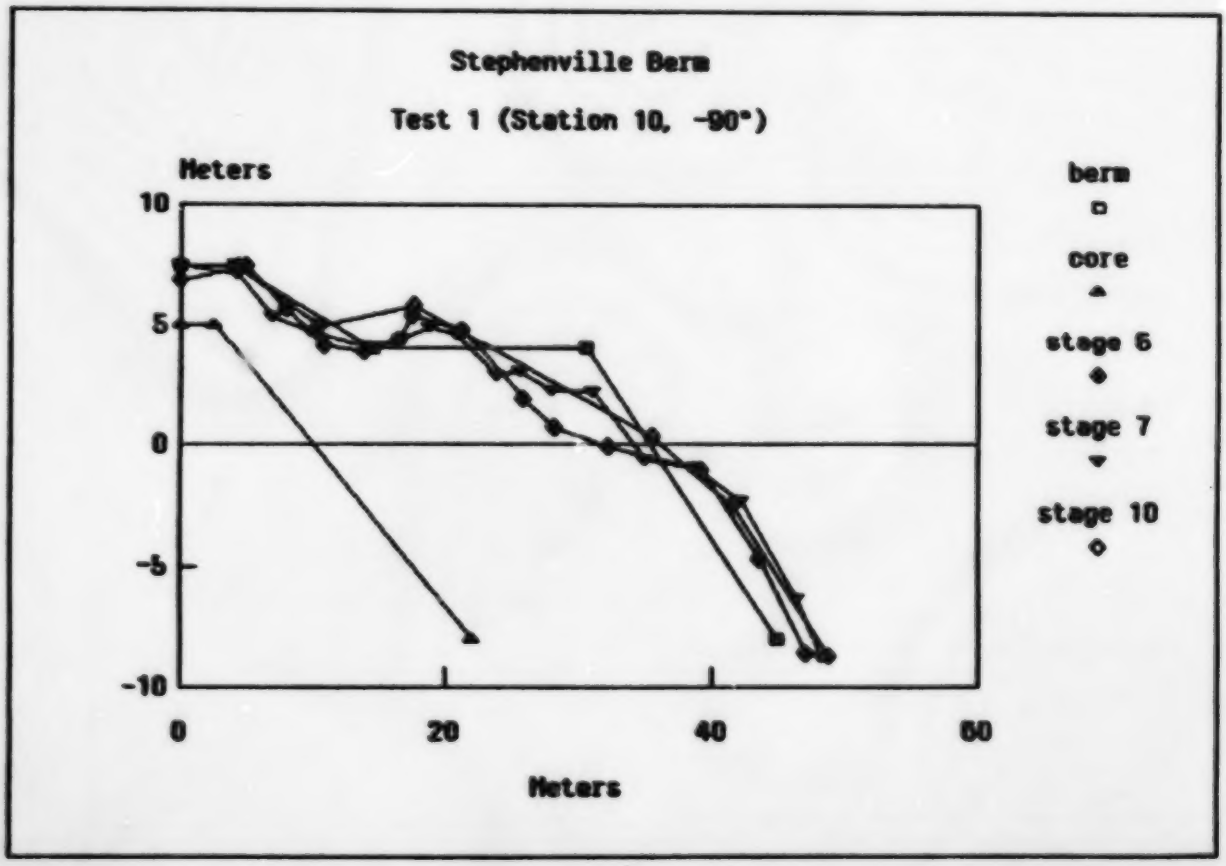

Stephenville Berm
Test 1 (Station +90°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10

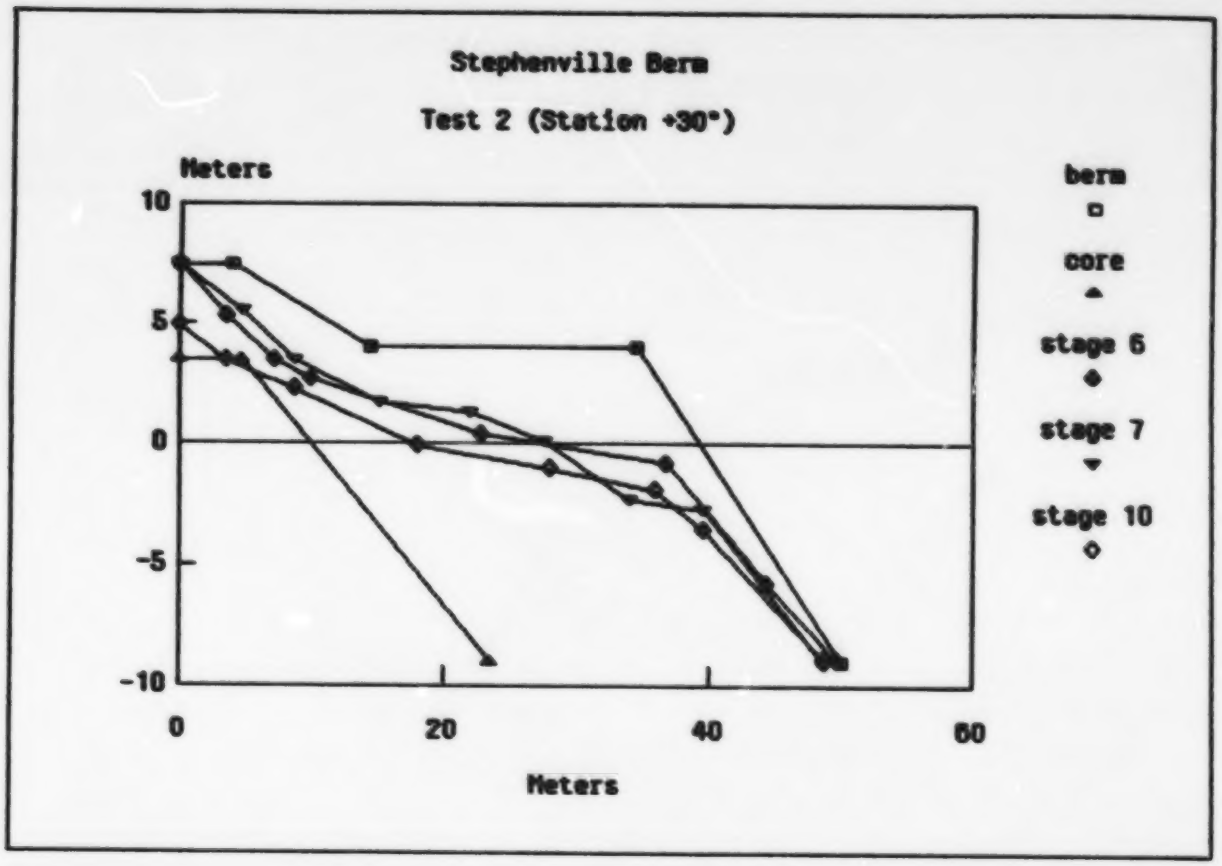

Stephenville Berm
Test 1 (Station 10, +90°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10

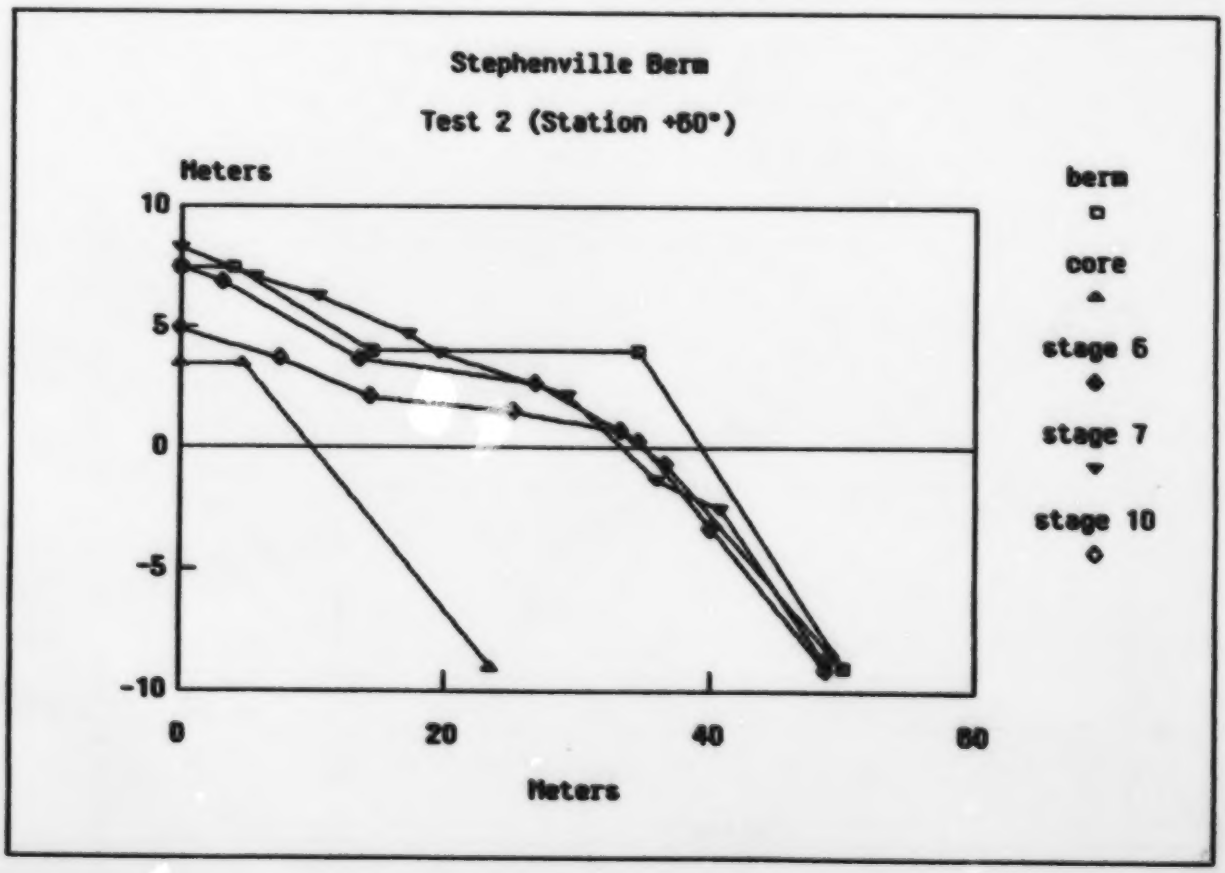

Stephenville Berm
Test 1 (Station -30°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10

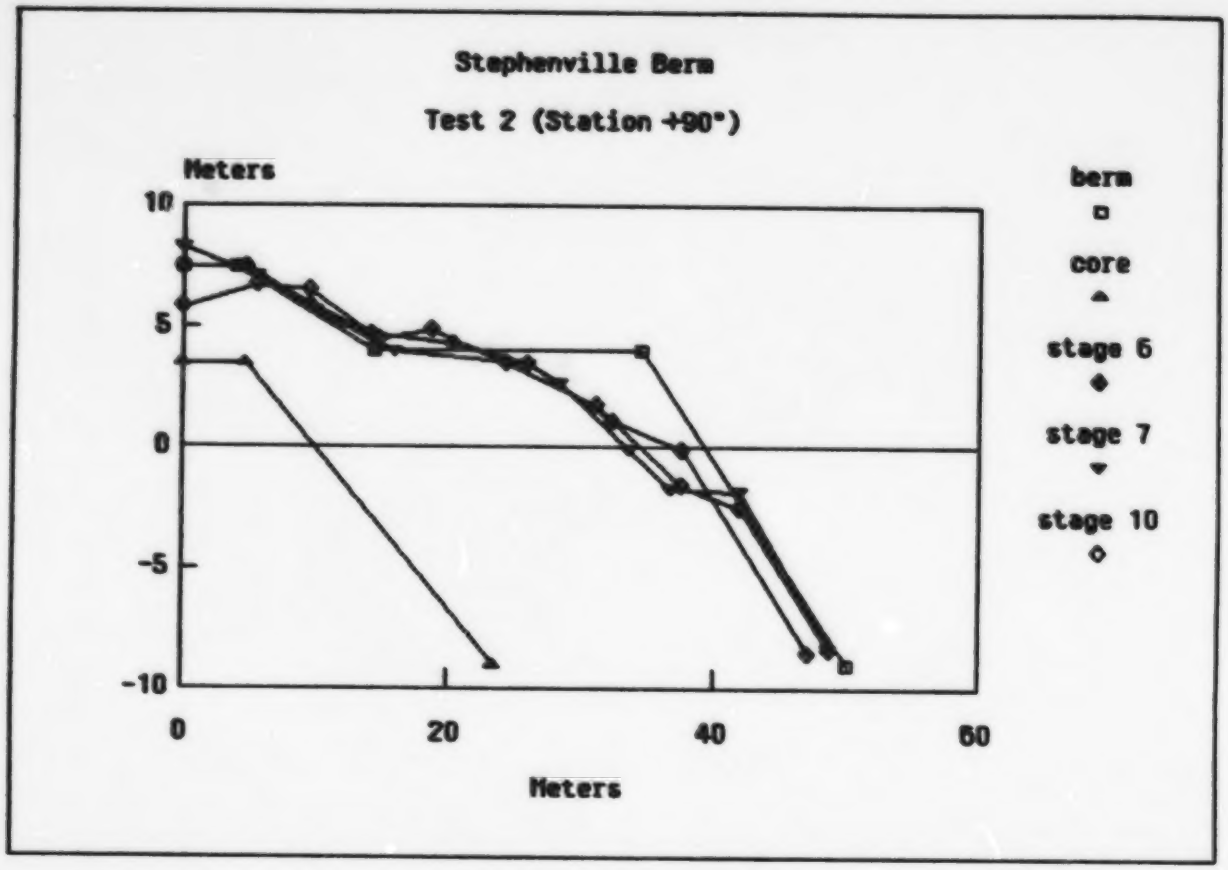

Stephenville Berm
Test 1 (Station -60°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10

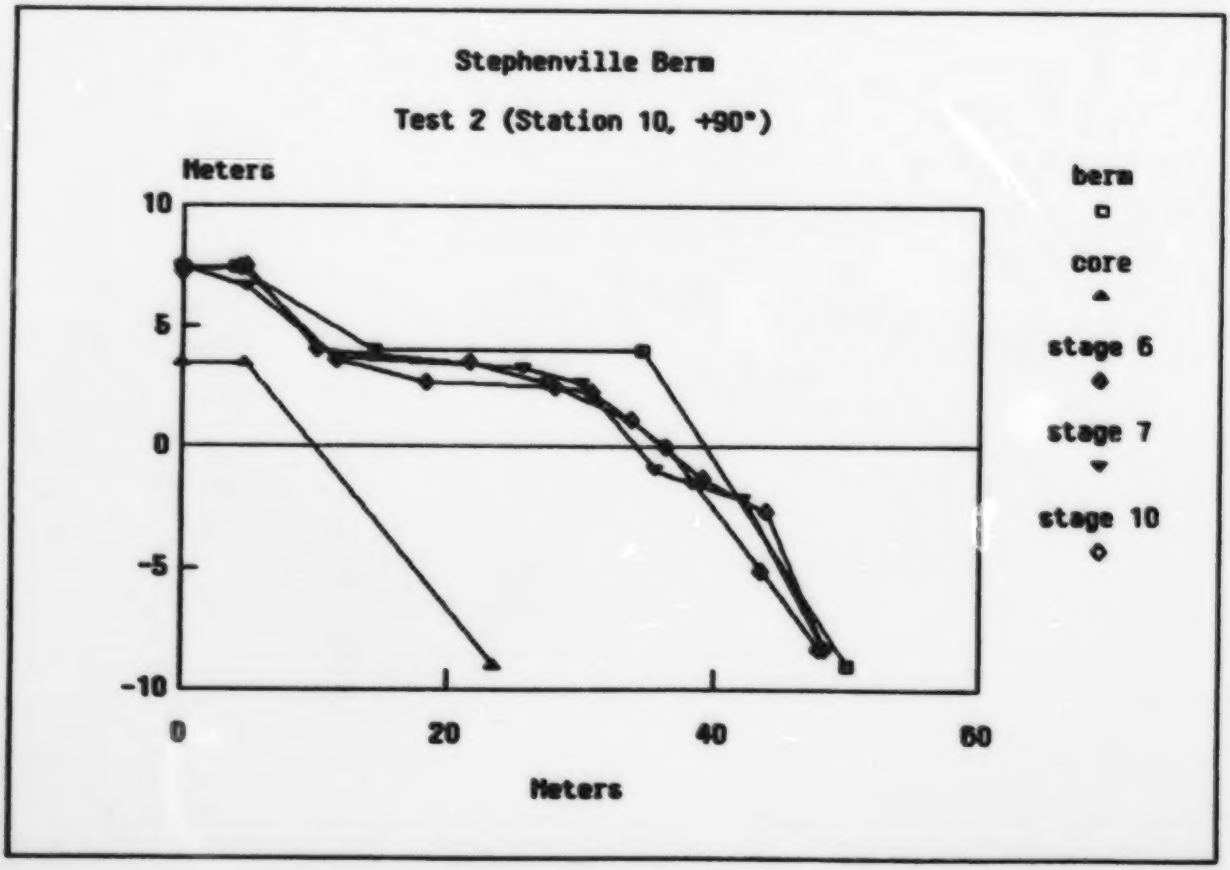

Stephenville Berm
Test 1 (Station -90°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10

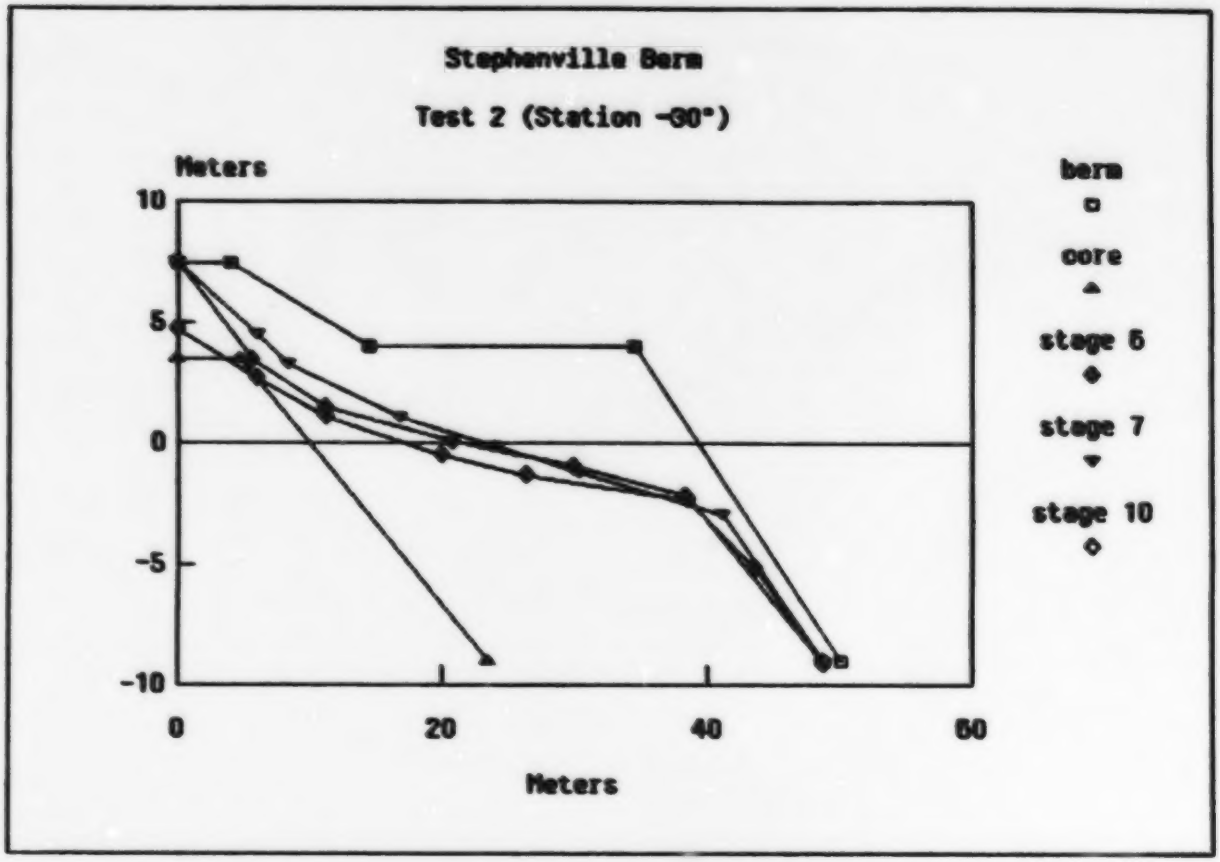

Stephenville Berm
Test 1 (Station 10, -90°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10

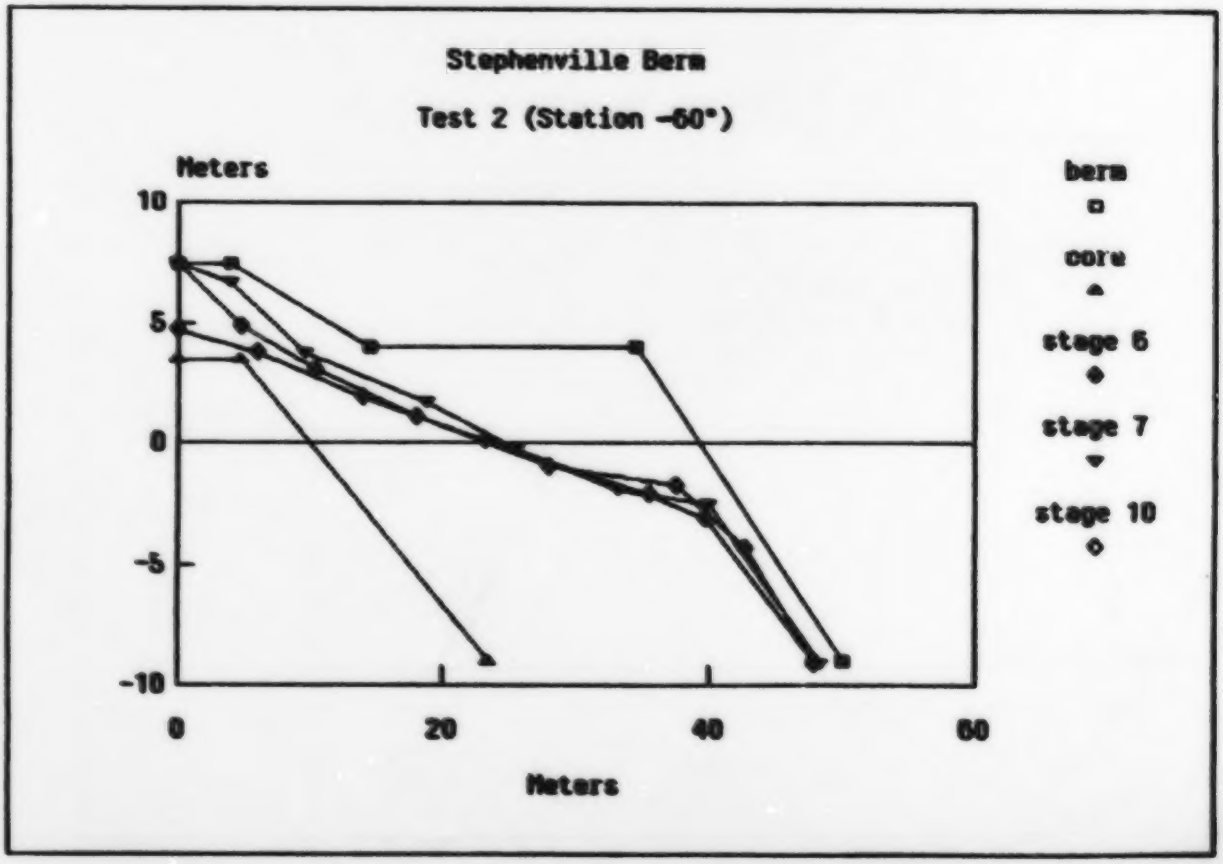

Stephenville Berm
Test 2 (Station +30°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10

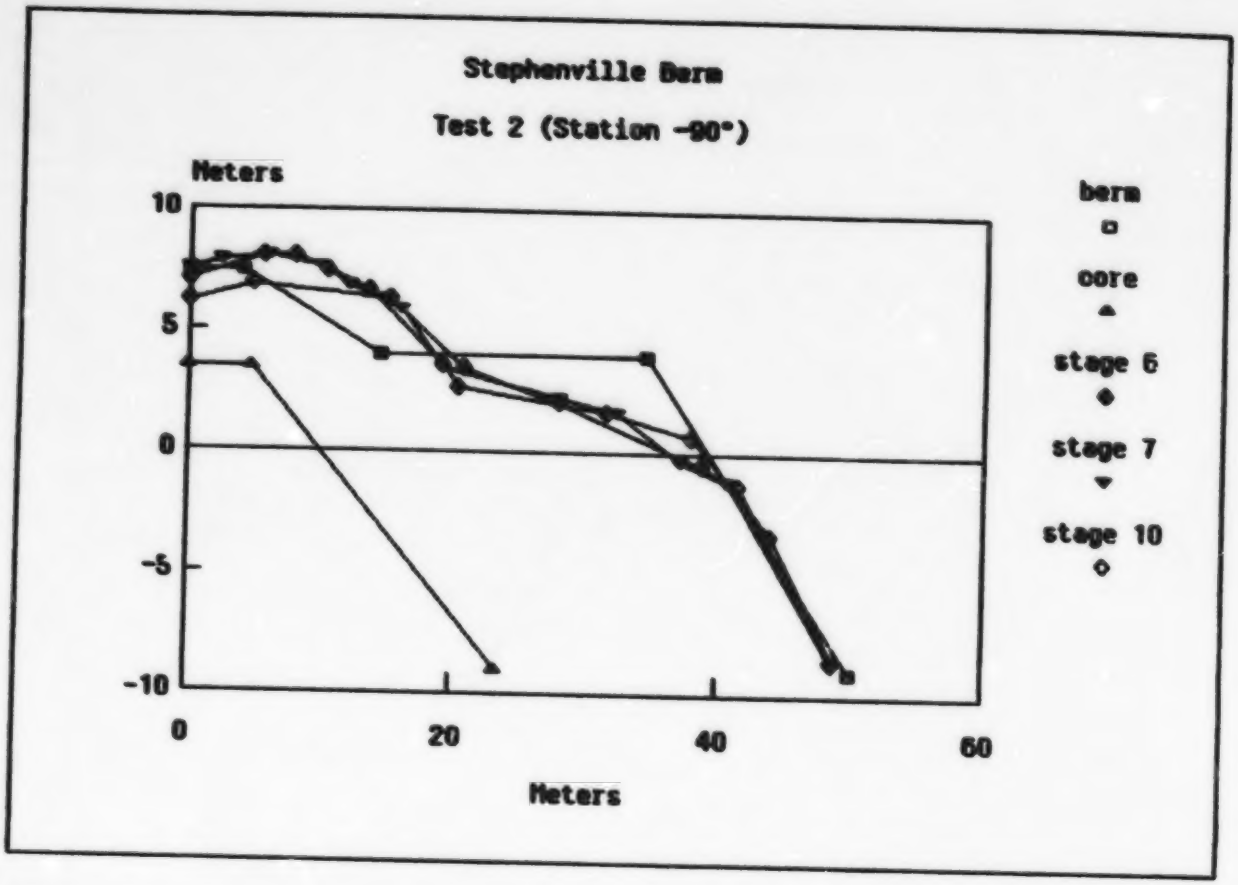

Stephenville Berm
Test 2 (Station +60°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10

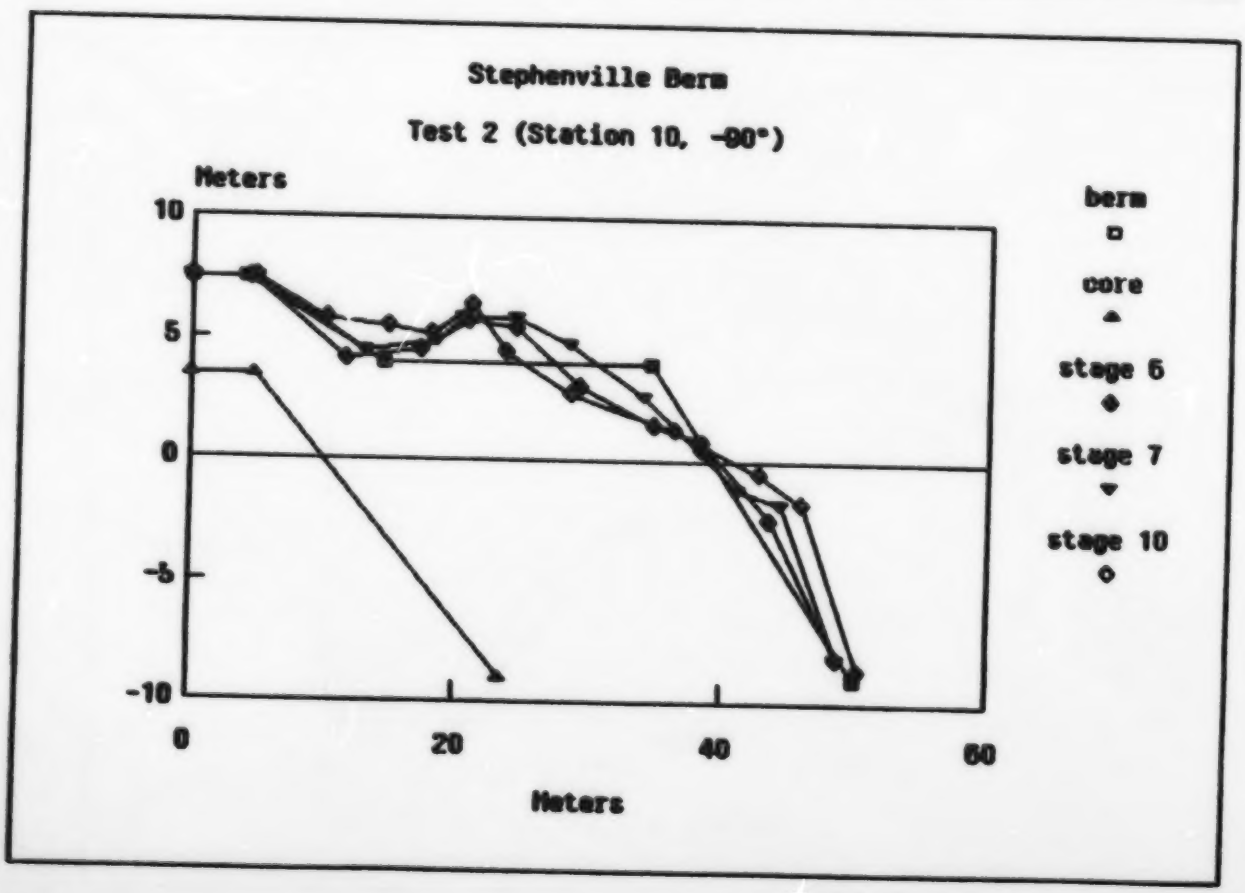

Stephenville Berm
Test 2 (Station +90°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10

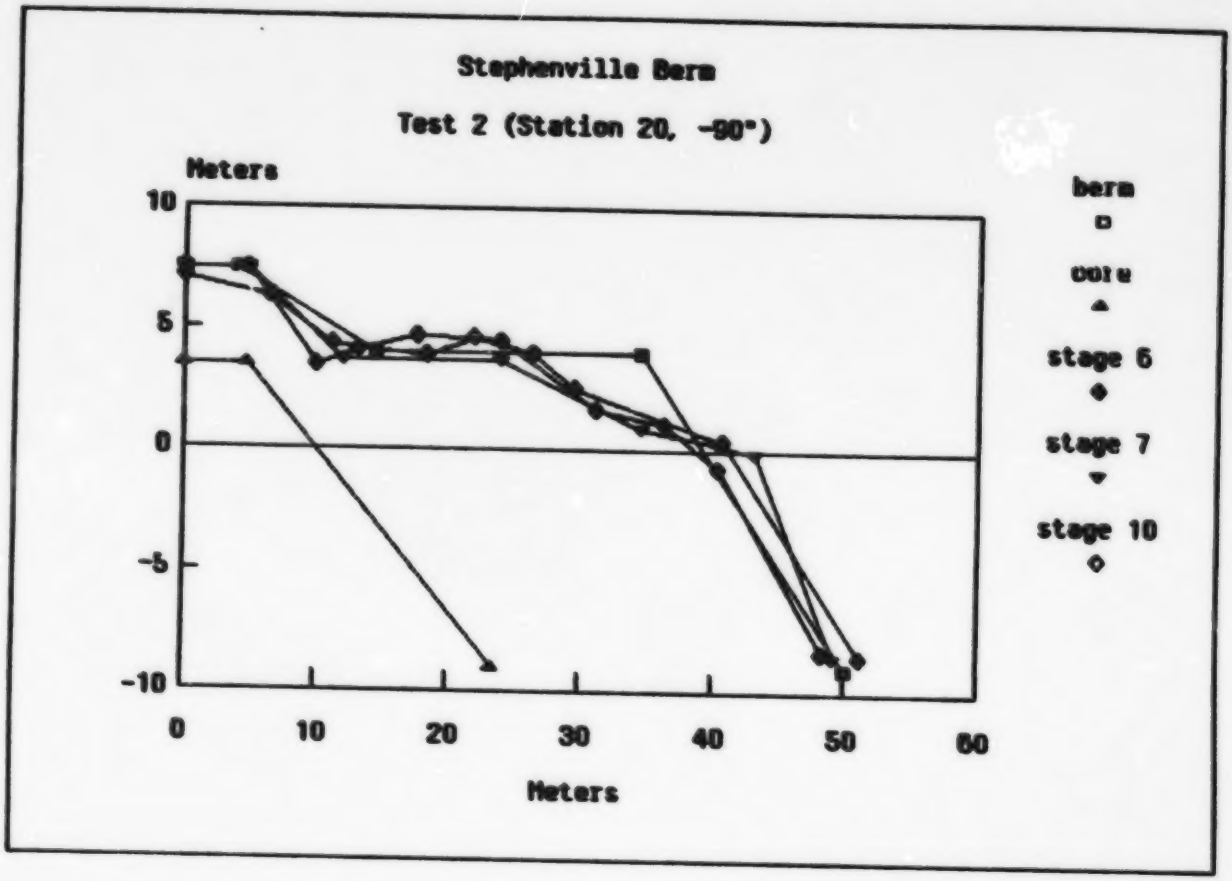

Stephenville Berm
Test 2 (Station 10, +90°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10

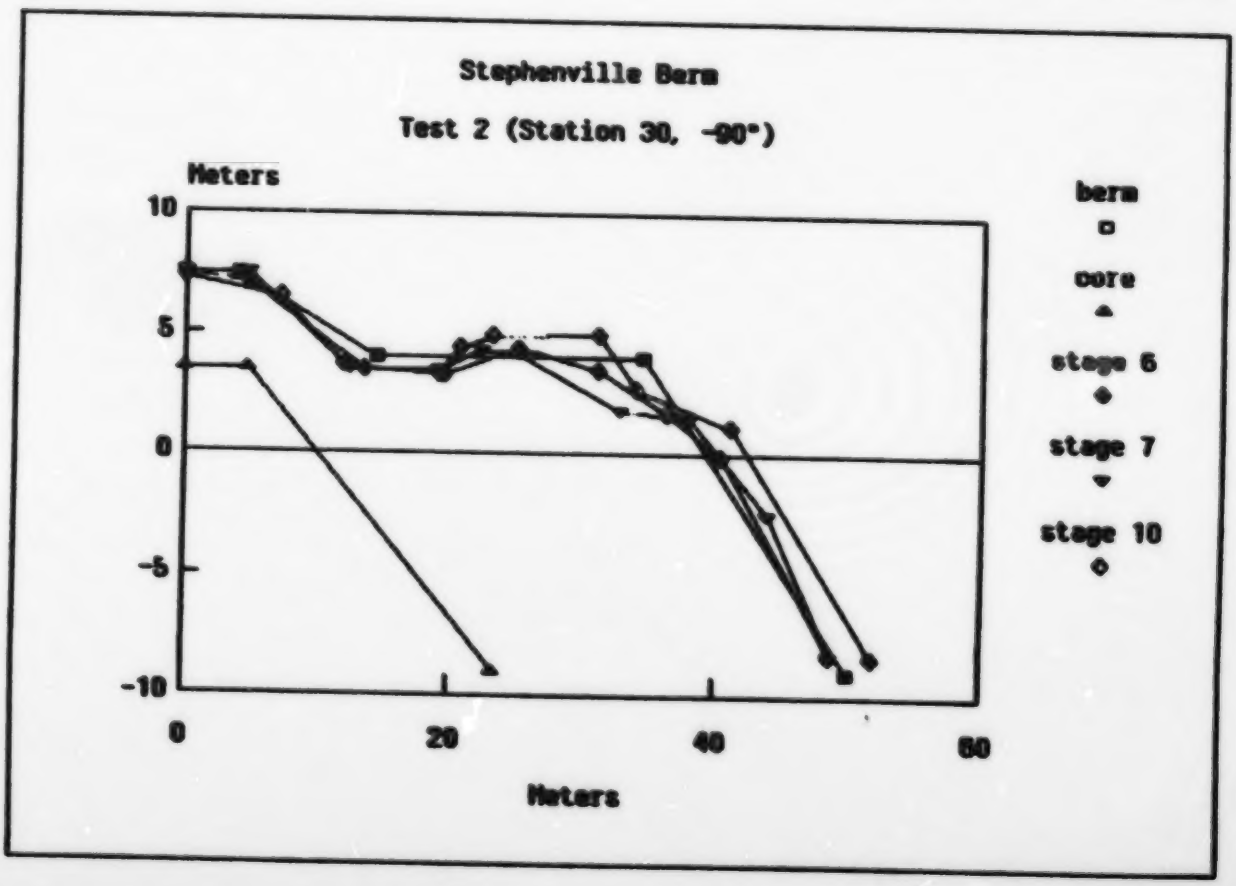

Stephenville Berm
Test 2 (Station -30°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10


Stephenville Berm
Test 2 (Station -60°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10


Stephenville Berm
Test 2 (Station -90°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10


Stephenville Berm
Test 2 (Station 10, -90°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10

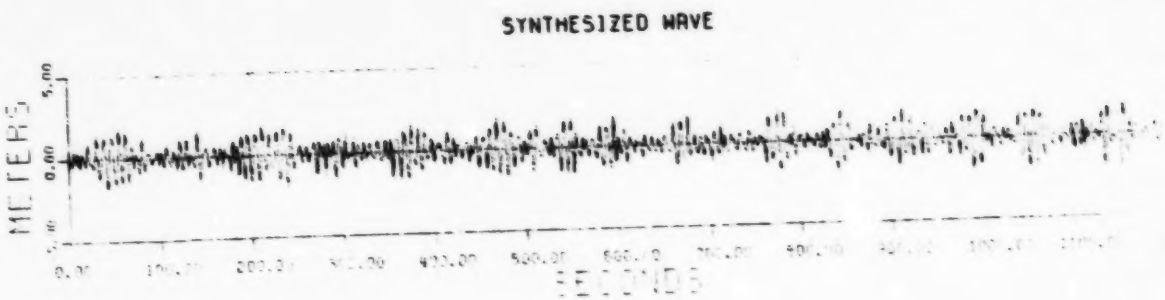

Stephenville Berm
Test 2 (Station 20, -90°)
Meters
10
5
0
-5
-10
0
10
20
30
40
50
60
Meters
berm
core
stage 6
stage 7
stage 10

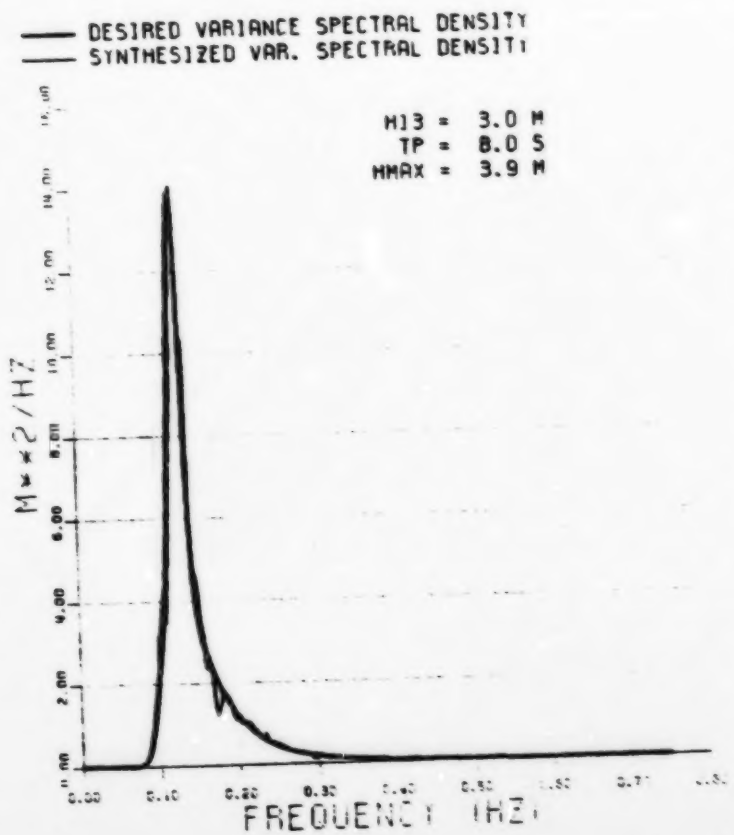

Stephenville Berm
Test 2 (Station 30, -90°)
Meters
10
5
0
-5
-10
0
20
40
60
Meters
berm
core
stage 6
stage 7
stage 10

APPENDIX D

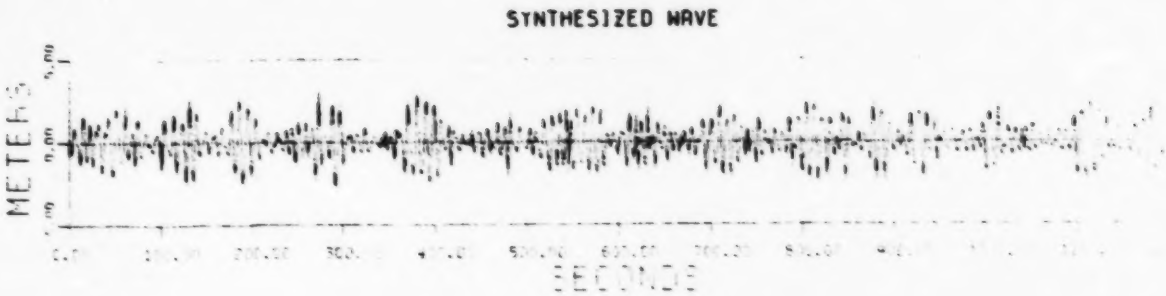
Irregular Wave Train and Variance Spectral Density diagrams for design storm stages 1, 2, 3, and 5

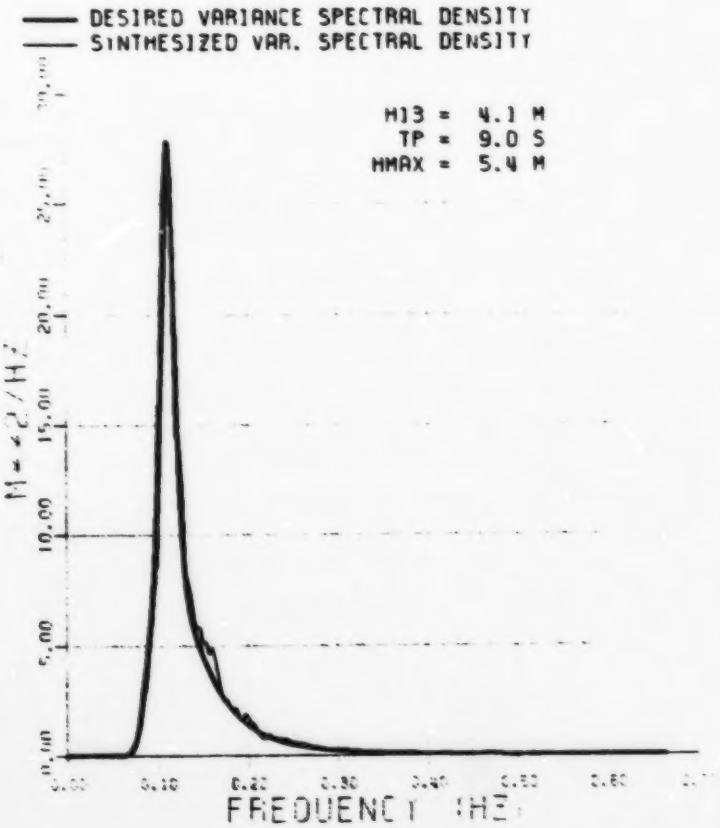
WAVE CHARACTERISTICS FOR STORM PROFILE STAGE 1



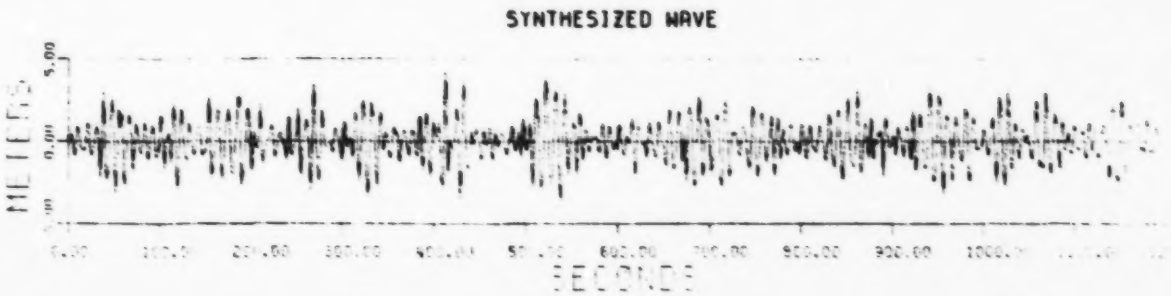


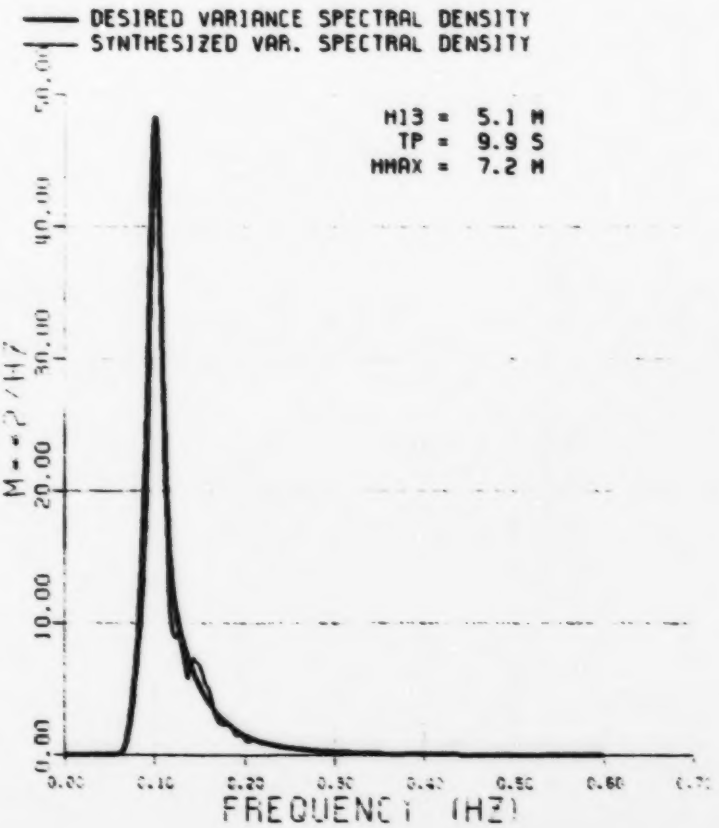
WAVE CHARACTERISTICS FOR STORM PROFILE STAGE 2



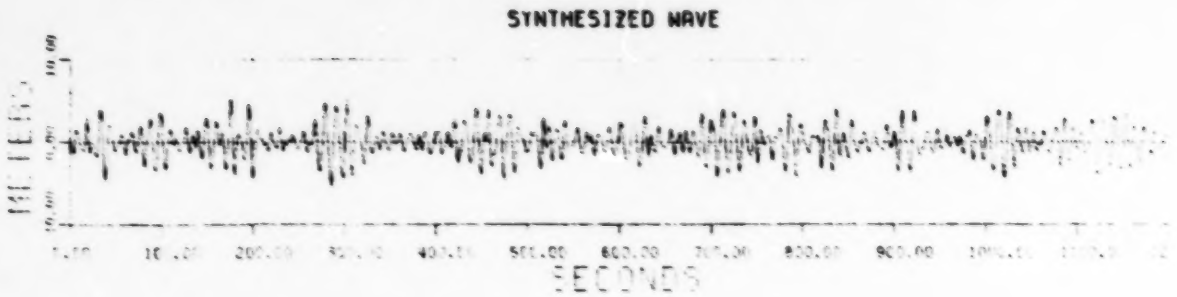


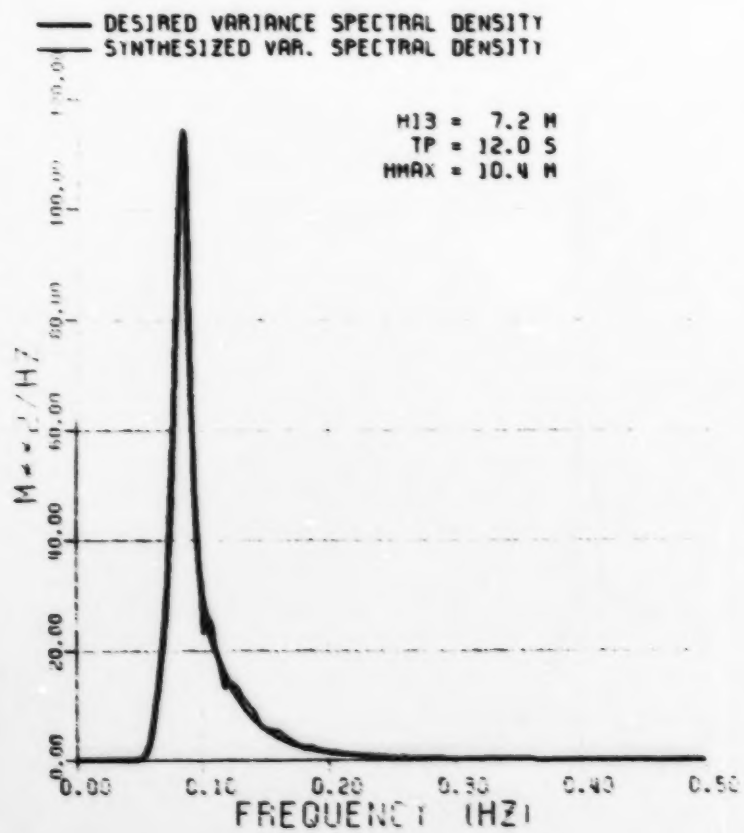
WAVE CHARACTERISTICS FOR STORM PROFILE STAGE 3





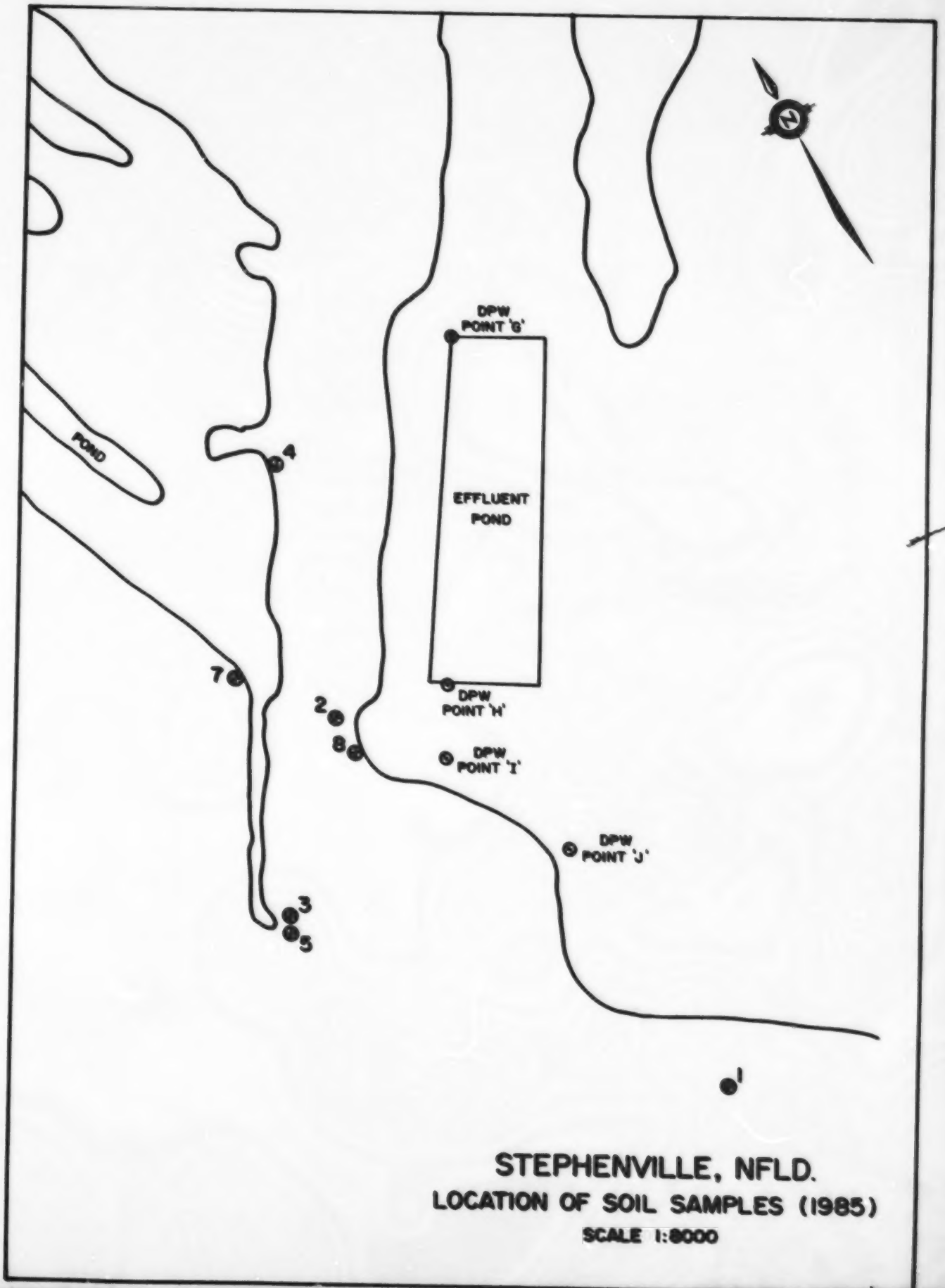
WAVE CHARACTERISTICS FOR STORM PROFILE STAGE 5





APPENDIX E

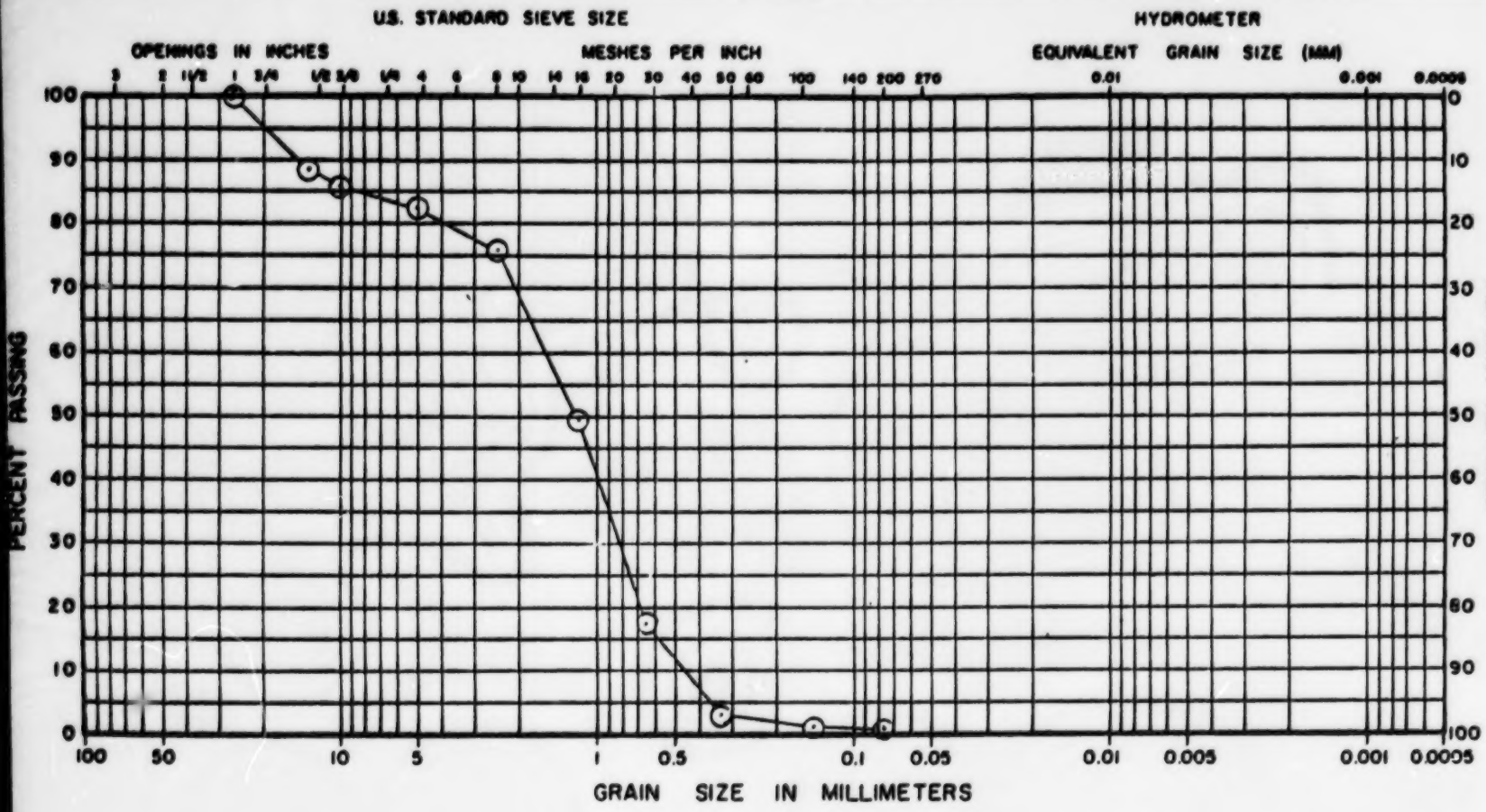
Sieve analyses of soil samples at various locations on the south beach and in the entrance channel



STEPHENVILLE, NFLD.

LOCATION OF SOIL SAMPLES (1985)

SCALE 1:8000



GRAVEL		SAND			SILT & CLAY
Coarse	Fine	Coarse	Medium	Fine	

UNIFIED SOIL CLASSIFICATION

BOREHOLE	TESTPIT	SAMPLE	DEPTH	DESCRIPTION	
1		1	0.0 - 0.5m	Sand; some gravel; trace silt.	SP

GRAIN SIZE DISTRIBUTION

APPENDIX B
FIGURE 1
PROJECT 84-310

OceanChem

Grain Size Analysis Sample 2

Sample Name Stephenville G851

PROJECT NO.: 5927-4

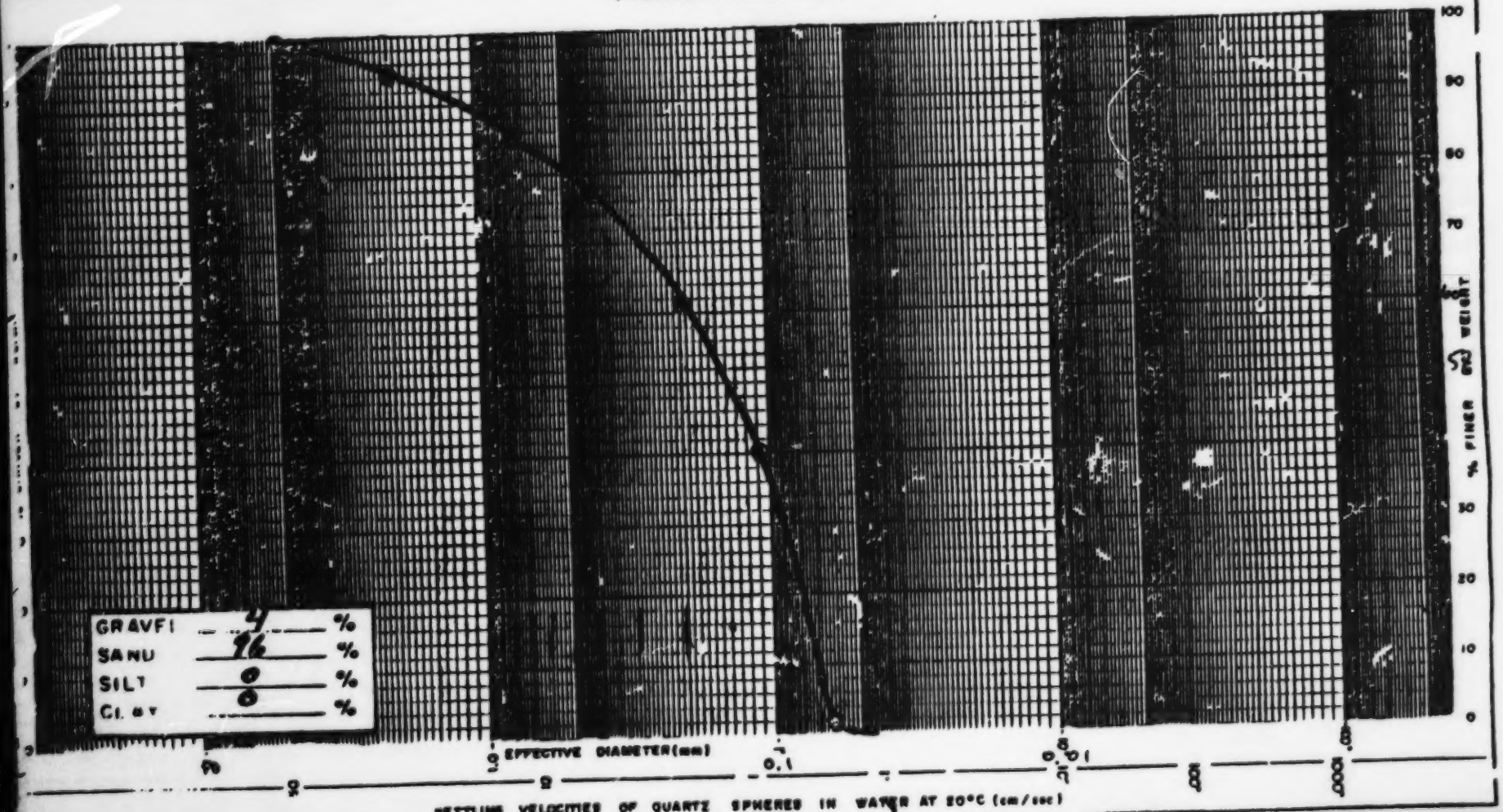
DATE: July, 1985

-5	-4	-3	-2	-1	0	1	2	3	PHI	4	5	6	7	8	9	10	11
		4000	2000	1000	500	250	125	63	METRIC SIEVES MESH								
	3	4	5	10	20	40	60	100	U.S. SIEVES MESH								

SIEVE ANALYSIS → ← PIPETTE ANALYSIS →

GRAVEL | SAND | SILT | CLAY

(WENTWORTH CLASSIFICATION)



SETTLING VELOCITIES OF QUARTZ SPHERES IN WATER AT 20°C (cm/sec)

Grain Size Analysis Sample 8

Sample Name Stephenville G855

OceanChem

PROJECT NO: 5927-4

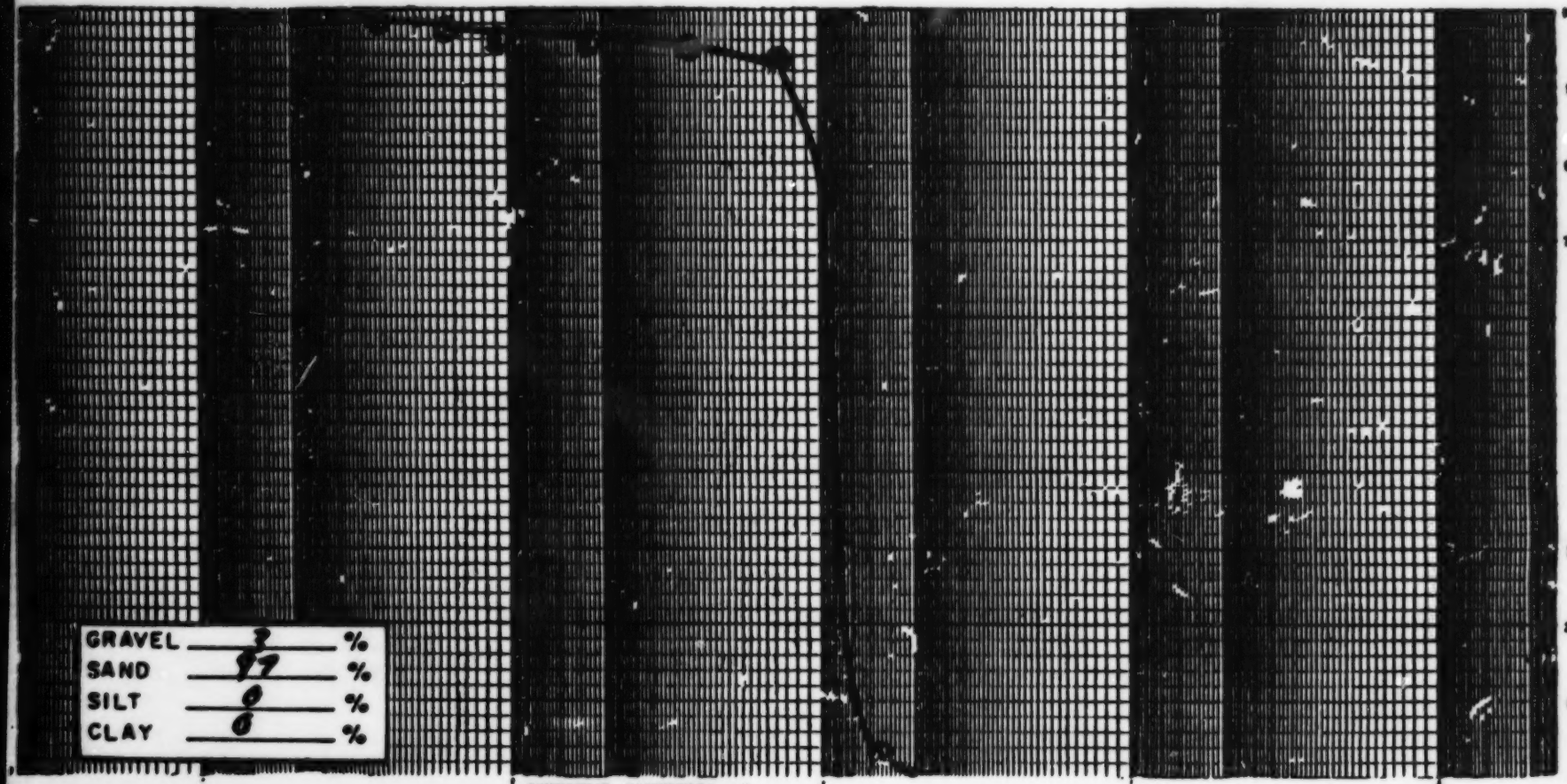
DATE: July, 1985

-5	-4	-3	-2	-1	0	1	2	3	PHI	4	UNITS	5	6	7	8	9	10	11
		9000	8000	7000	6000	5000	4000	3500	3150	75	METRIC SIEVES MESH							
	3	4	5	10		20	40	60	100	200	U.S. SIEVES MESH							

← SIEVE ANALYSIS → ← PIPETTE ANALYSIS →

GRAVEL | SAND | SILT | CLAY

(WENTWORTH CLASSIFICATION)



Golder Associates

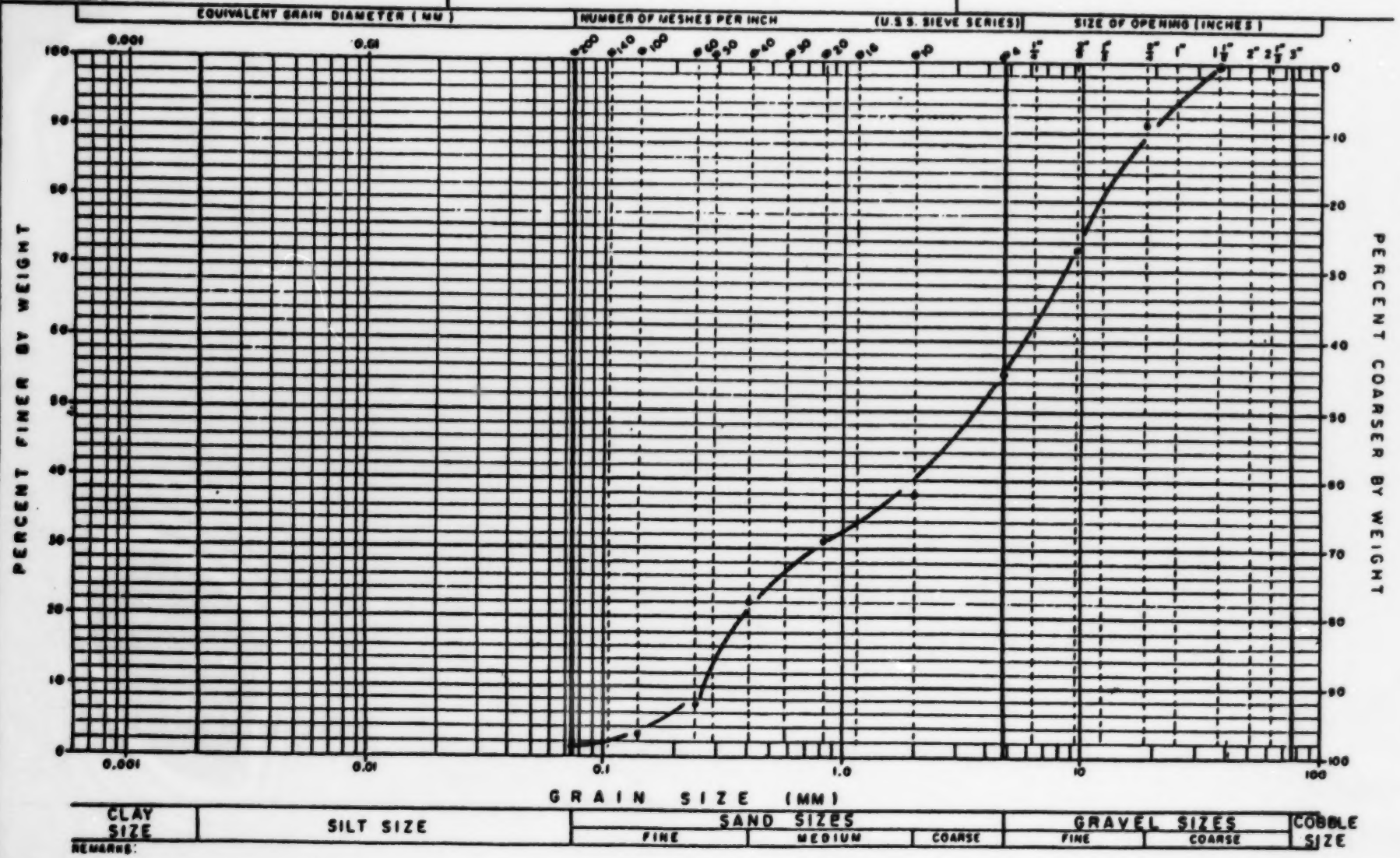
GRAIN SIZE DISTRIBUTION

SOIL SAMPLE DESCRIPTION: (UNIFIED SOIL CLASSIFICATION)

SAND and GRAVEL, trace silt

Stephenville Harbour Improvements

Sample No. 4



REMARKS:

Golder Associates

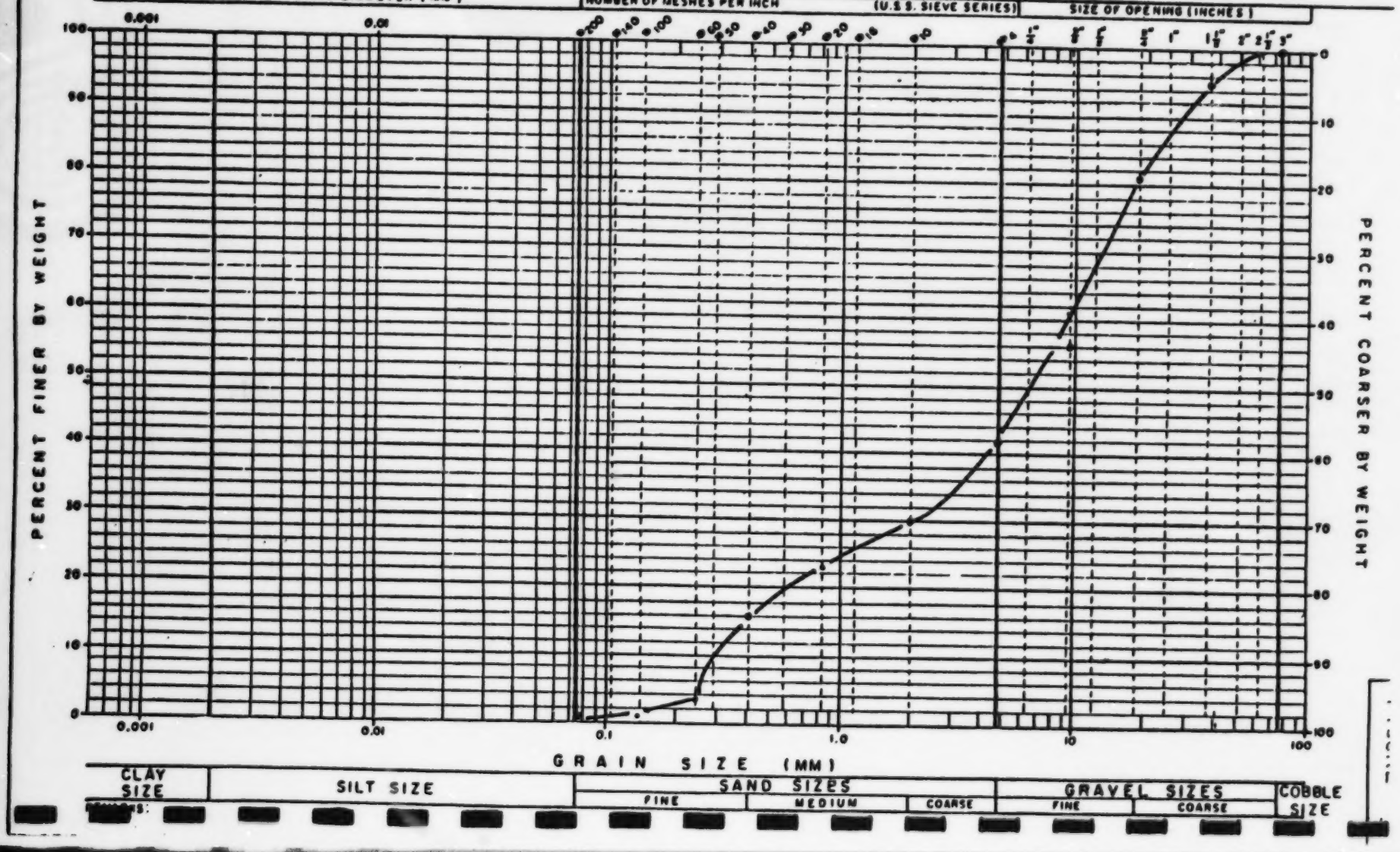
GRAIN SIZE DISTRIBUTION

SOIL SAMPLE DESCRIPTION: (UNIFIED SOIL CLASSIFICATION)

SAND and GRAVEL, trace silt

Stephenville Harbour Improvements

Sample 6



PROJECT NO. 03[illegible]

Golder Associates

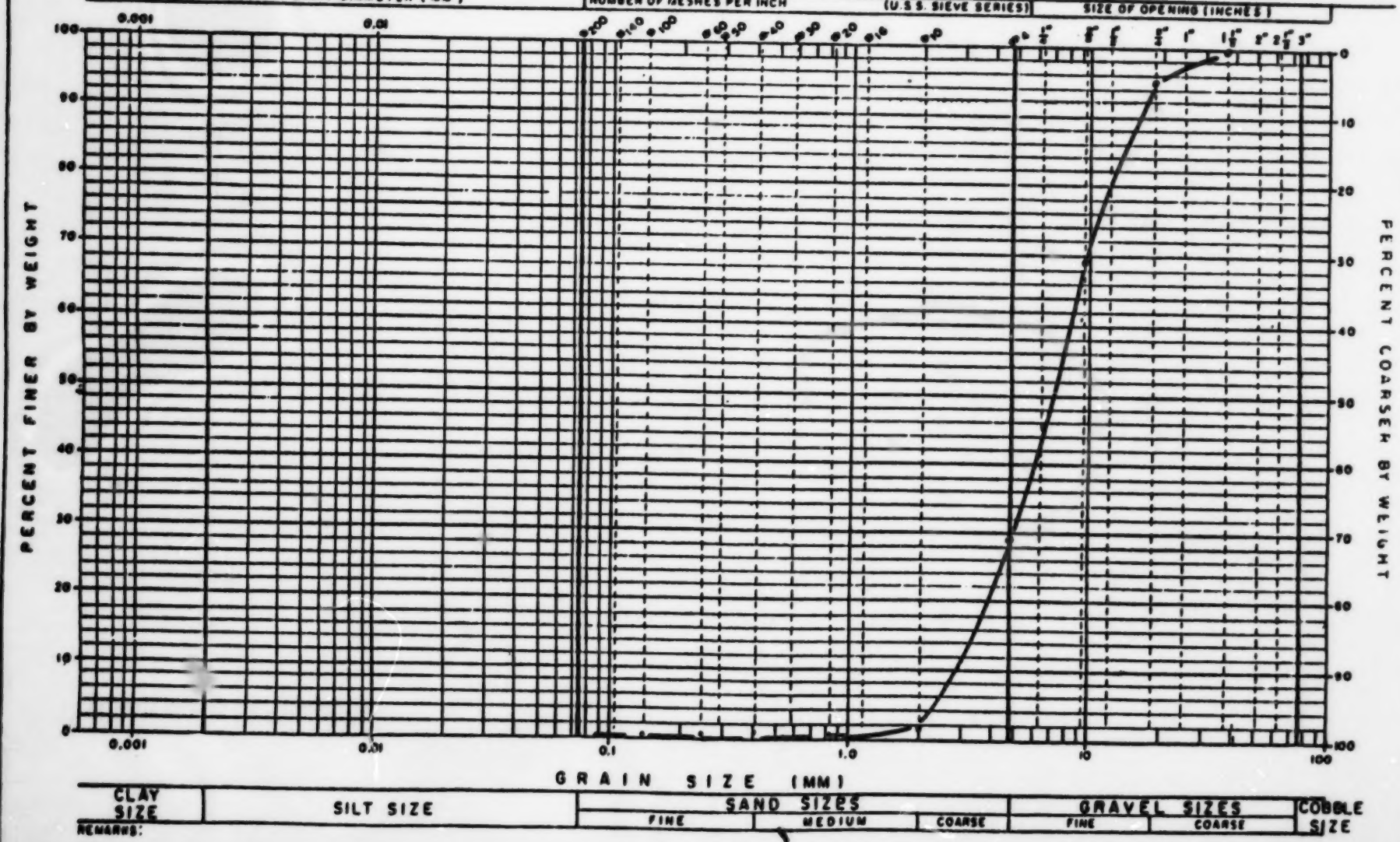
GRAIN SIZE DISTRIBUTION

SOIL SAMPLE DESCRIPTION: (UNIFIED SOIL CLASSIFICATION)

Sandy GRAVEL

Stephenville Harbour Improvements

Sample No. 7



REMARKS:

PROJECT NO. 021-7117

GRAIN SIZE DISTRIBUTION

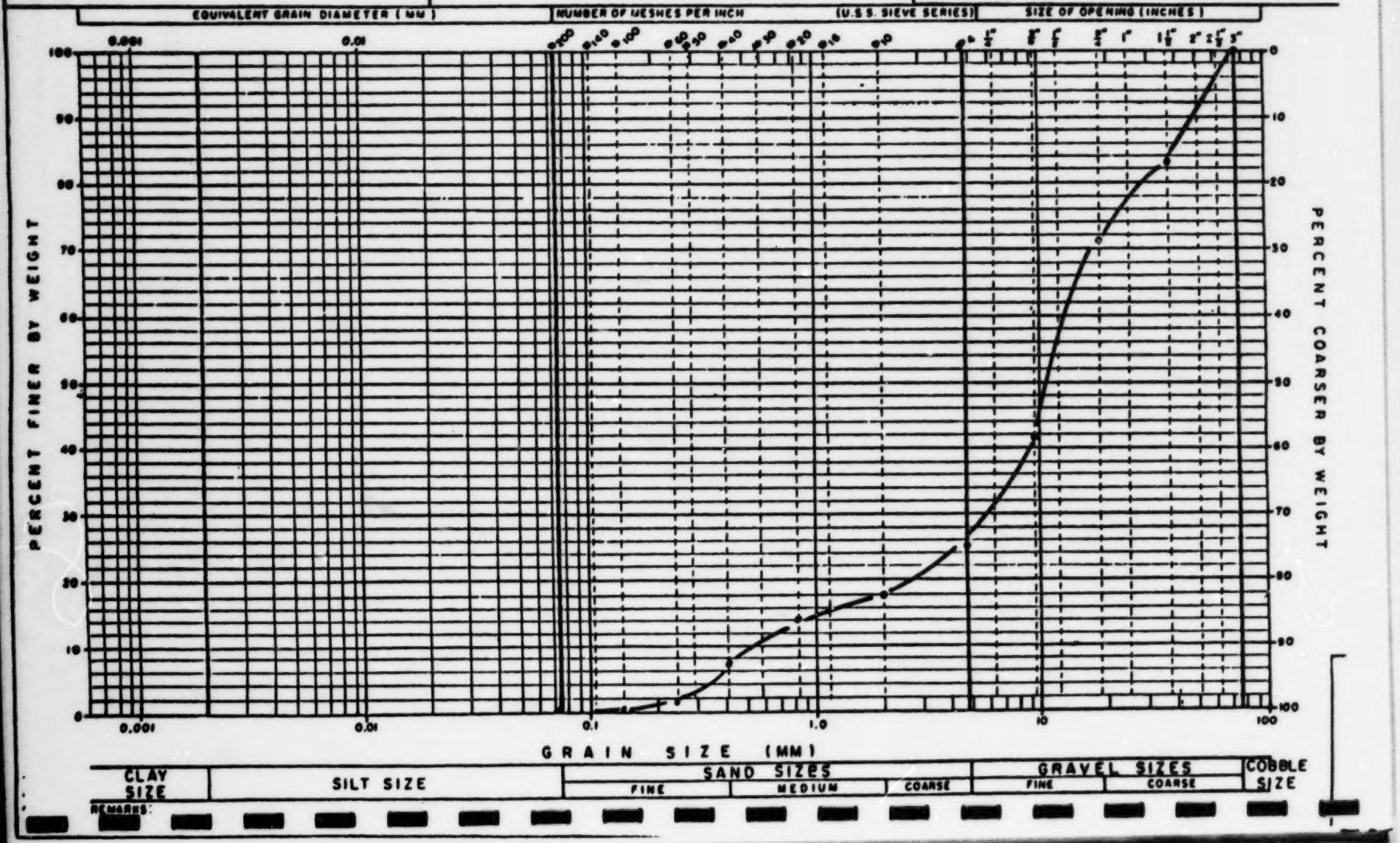
Golder Associates

SOIL SAMPLE DESCRIPTION: (UNIFIED SOIL CLASSIFICATION)

Sandy GRAVEL

Stephenville Harbour Improvements

Sample No. 8



REMARKS: